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AN INVESTIGATION OF THE LeSUEUR LANDSLIDE
EDMONTON, ALBERTA

by

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The undersigned certify that they have read, and recommend to the Faculty of Graduate Studies for acceptance a thesis entitled "AN INVESTIGATION OF THE LeSUEUR LANDSLIDE, EDMONTON, ALBERTA", submitted by WILLIAM THOMAS PAINTER in partial fulfilment of the requirements for the degree of Master of Science.

ABSTRACT

The purpose of this research study has been the investigation and analysis of the LeSueur Landslide which occurred in 1963, in the valley of the North Saskatchewan River near Edmonton, Alberta.

Site investigation using various drilling techniques was used to establish the soil profile. Piezometers and probewells were also installed. Laboratory testing employed ASTM procedures and shear strength parameters were determined with the Geonor apparatus using the consolidated undrained triaxial test with pore pressure measurements. A review of relevant case histories was made.

The soil profile is complex and consists of glacial valley-infillings overlying the Upper Cretaceous Edmonton Formation. Significant proportions of the clay mineral montmorillonite are present in the profile soils. Laboratory evidence suggests that these soils are slightly overconsolidated and their behavior appears to be influenced by the clay minerals present.

It is postulated that the LeSueur Landslide was initiated by an annual process of toe erosion by the river during the summer months followed by pore water pressure build up during the winter months. The geometry of the landslide resembled that of a sliding block having its base within a bentonitic layer of the Edmonton Formation.

Stability analyses in terms of effective stress gave the following results:

(a) A simplified sliding block analysis gave a factor of safety of 0.97 assuming the base of the landslide was in a layer of bentonite.

(b) Analyses according to Bishop (1954) gave factors of safety of 0.92 assuming a slip circle closely approximating the shape of the slip surface tangential to a layer of bentonitic clay shale and 1.13 assuming the same slip circle tangential to a layer of bentonite.

It is recommended that further physico-chemical analyses and shear strength tests be performed on undisturbed samples of the LeSueur site soils which may show relationships between the shear strength characteristics, the clay mineralogy and type of exchangeable cations.

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GLOSSARY OF TERMS AND SYMBOLS

TERMS

Activity - Defined by Skempton (1953) as the ratio of the Plasticity Index to the percent clay sizes for a cohesive soil.

$$\text{i.e. Activity} = \frac{\text{Plasticity Index}}{\text{Clay Fraction}}$$

Inactive clays - Activity 0.75
Normal Clays - Activity 0.75 to 1.25
Active Clays - Activity 1.25

Natural Slopes - Man-made slopes or slopes formed by geological processes which are in equilibrium under the existing stress conditions and are influenced only by changes in groundwater level.

Overthrust - The section of the slip surface which protrudes above the original ground surface.

Sensitivity - The sensitivity of a clay is defined as the ratio of its undisturbed to its remoulded strength at the same moisture content.

SYMBOLS

W_L	Liquid limit
W_P	Plastic limit
w_S	Shrinkage limit
w	Natural moisture content
I_P	Plasticity index
A	Activity
c'	Effective cohesion

ϕ'	Angle of shearing resistance with respect to effective stress
$\bar{c}'$	Average effective cohesion
$\bar{\phi}'$	Average effective angle of shearing resistance
σ'_{1f}	Major principal effective stress at failure
σ'_{3f}	Minor principal effective stress at failure
$(\sigma_1 - \sigma_3)_f$	Deviator stress at failure
u_f	Pore pressure at failure
$B = \frac{\Delta u}{\Delta \sigma_3}$	Pore pressure parameter B
q_u	Unconfined compressive strength
F	Factor of safety with respect to shear strength
S_r	Degree of saturation
ASTM	American Society for Testing Materials
PFRA	Prairie Farm Rehabilitation Administration - Soil Mechanics Division
USBR	United States Bureau of Reclamation

CHAPTER I

INTRODUCTION

1.1 Slope Stability Concepts

The engineer is concerned with two main classes of slope stability problems which depend on the field conditions. These may be termed "short-term" stability problems and "long-term" stability problems. "Short-term" problems are those which comprise man-made slopes or excavations where the pore pressure in the soil involved has not had sufficient time to adjust itself to the stress changes caused by the construction operation. "Long-term" problems comprise natural* slopes in which the pore pressure in the soil has had time to reach equilibrium under the existing stress conditions and is affected only by changes in groundwater level.

The shear strength parameters required for the analysis of these two classes of problems depend upon the stress conditions. In "short-term" stability problems, the stresses may be either total or effective. For example, a short time after the construction of an excavation there is insufficient time for transfer of stress from the fluid phase to the solid phase within the soil mass and the stresses determining the shear strength of the soil are total stresses. However, in "short-term" problems concerning the construction of embankments, the gravity stresses may cause pore pressures within the embankment, as the construction proceeds. In

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*Words or phrases marked thus are defined in alphabetical order in the Glossary of Terms and Symbols.

this class of "short-term" problem the stresses are effective stresses. In "long-term" stability problems, other than in exceptional circumstances, the analysis is required in terms of effective stresses which necessitates taking into account stresses in both the fluid phase and the solid phase.

1.2 Existing Knowledge of Stability Analysis of Natural Slopes

Many natural slope failures in various soil types have been reported in the literature. Generally, the factor of safety of these has been determined for the time of failure using the shear strength of soil samples recovered from near the slip surface. At the time of failure, the factor of safety must have been unity since, for limiting equilibrium, the shear stresses set up in this zone were equal to the available shear strength.

Total stress analyses have been widely used in the past because of their simplicity, but it has now been generally accepted that these analyses are not usually valid for the solution of "long-term" stability problems. Those effective stress analyses performed, using the peak shear strength parameters of the soil and measured field pore water pressures, have given suitable factors of safety for some normally consolidated clays and overconsolidated clays with intact structures. (Bjerrum and Kjaernsli, 1957, Bishop and Bjerrum, 1960)[1]. However, in general, the factors of safety using these analyses for heavily overconsolidated fissured clays have been too high. In relation to the stability analysis of natural slopes, one of the hypotheses is that the effect of fissures is to reduce the

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1. References are listed alphabetically by author in the List of References at the end of this thesis.

cohesion of these soils along the potential slip surface. Modified effective stress analyses, in Europe, using zero cohesion, with the peak angle of shearing resistance, have given factors of safety closer to unity. (Henkel and Skempton, 1955, Henkel 1957).

For the highly overconsolidated clay shales of western Canada, Hardy, Brooker and Curtis (1962) have performed analyses which take account of the swelling pressures of these soils. They have obtained factors of safety close to unity for failed slopes by reducing the effective normal stress on the potential slip surface by an amount equal to the swelling pressure of the soil. Recently, Ringheim (1964) has suggested that failures of natural slopes in these clay shales are the result of reactivation of ancient slip surfaces upon which the shear strength is the residual value. Skempton (1964) has endorsed this opinion and has postulated that the length of slip surface over which the residual strength is developed varies with the gross structure of the overconsolidated clay. Skempton has re-examined earlier effective stress analyses in Europe, using residual shear strength parameters but has reported that to the present time the data available for natural slopes is inadequate.

1.3 Occurrence of the Landslide

A landslide, which is typical of many which occur along the banks of the North Saskatchewan River, occurred in the Edmonton area in 1963, and by virtue of its proximity to the University of Alberta, provided a convenient opportunity for research studies. The geology of the Edmonton area is typical of much of the plains area of western Canada and consists of glacial deposits underlain by highly overconsolidated fissured clay

shales interbedded with sandstones. This particular landslide has been termed the LeSueur Landslide for the purposes of this thesis. It is located on the National Topographic Map series 83/11,W1/2, Edmonton Sheet at latitude 53°36'05" and longitude 113°18'40". (Legal Subdivision 3, Section 27, Range 23, Township 53, West of the 4th Meridian). This is about five miles northeast of Edmonton on the outside of a meander bend of the North Saskatchewan River. A location map is given in FIGURE 1, and an aerial view of the landslide area in PLATE 1.

The appearance of tension cracks along the top of the river bank at this location were observed and reported by Mr. R. LeSueur in January 1963. Slow vertical movements were noted from this time on, and a scarp of about 2 ft. had developed by July 1963. These slow movements continued until August 31st., 1963, when the river bank slipped at a rate of about 2 ft. per hour to a level of 30 ft. below its original position, undermining a corner of Mr. LeSueur's house. (See PLATE 2). Mr. LeSueur observed that the movement at the toe appeared to take place along a horizontal layer and that the moving soil mass fell off over the original river bank. The author found by later observations that the bank had been extended riverward about 40 ft. Although the slope was thickly wooded, the trees contributed nothing to the stability and their roots were easily snapped off as the movement took place.

1.4 Purpose of the Research

The purpose of the research study on which this thesis is based is the investigation and analysis of this particular failure of the river bank. Various analytical methods are used to ascertain which appears to

best fit the field conditions. The geological and hydrological factors influencing the landslide are considered and stability analyses are performed with respect to the shear strength of the soils involved in the failure. Both an average slope through the landslide area and an adjacent stable slope are investigated.

1.5 Résumé of the Study

This thesis comprises three main sections; the site investigation reported in CHAPTER III, the laboratory testing program described in CHAPTER IV, and the stability analyses which are presented and discussed in CHAPTER VI. For ease of interpretation, these sections of the study are summarised;

A. SITE INVESTIGATION

The site investigation included;

(a) A topographical survey to ascertain the geometry of the landslide.

(b) A subsurface exploration to determine the subsoil profile and to recover suitable samples for laboratory testing.

(c) Installation of piezometers to measure pore water pressure fluctuations in the river bank.

(d) Installation of probewells to determine the position and shape of the slip surface.

B. LABORATORY TESTING

Laboratory tests were performed for;

(a) Classification of the profile soils.

(b) Determination of the shear strength, physico-chemical properties

and mineralogy of representative samples of the soils involved in the failure.

(c) Examination of the chemical composition of the groundwater.

C. STABILITY ANALYSES

Stability analyses were performed using several methods to determine factors of safety with respect to shear strength for;

(a) An average section through the landslide area for the time of its failure.

(b) An adjacent stable slope.

The field and laboratory data are discussed in CHAPTER V. The conclusions arising from the research are given in CHAPTER VII, and recommendations for future studies concerning the LeSueur Landslide are included in CHAPTER VIII.

CHAPTER II

A REVIEW OF SOME RELEVANT CASE HISTORIES

2.1 Scope

The purpose of this chapter is to review a few of the available stability studies which appear to be relevant to the LeSueur Landslide analysis. These studies generally concern analyses of natural slopes and, in describing them it is convenient to consider the clays concerned under three headings; normally consolidated clays, overconsolidated clays with intact structures and overconsolidated fissured clays.

The most important aspect of a slope stability analysis is the selection of the shear strength parameters for the soils involved in the failure. In this regard, some consideration is given to the parameters adopted in the studies and, where possible, the test methods by which they are obtained. Additionally, the important aspects of the site investigations are examined.

2.2 Slopes in Normally Consolidated Clays

The first effective stress analysis of a natural slope in normally consolidated clays, reported in the literature, was that described by Skempton (1945). This failure occurred along the bank of a man-made meander cut-off near the mouth of the river Ouse, near Kings Lynn, Norfolk, England. The bank had been excavated, more than a hundred years earlier, to a depth of 25 ft. at a slope of 2.75:1 (20°), and comprised interbedded

deposits of post-glacial silts and silty clays which overlay beds of soft plastic clay and sandy gravel, respectively. From site observations, it was concluded that the major part of the slip surface was within the soft plastic clay and trial and error methods were used to locate a slip circle for the stability analyses. Peak shear strength parameters, in terms of effective stresses, were determined for all the cohesive soils in the bank by means of drained shear box tests. No actual measurements of pore water pressures were made at the site since these were considered to be controlled by tidal fluctuations. The factor of safety, with respect to effective stresses, for the assumed slip circle was found to be 1.15. A total stress analysis, using strengths from undrained triaxial tests, gave a factor of safety of 1.30 using the same slip circle.

Between the years 1954 and 1957, many stable slopes and landslides along the banks of several rivers in Norway were analysed in terms of total and effective stresses, by engineers at the Norwegian Geotechnical Institute. This very significant work was reported by Bjerrum and Kjaernsli (1957). For the total stress analyses, triaxial tests were used in conjunction with field vane tests to obtain undrained strengths. For the effective stress analyses, piezometers were installed to measure the pore pressures and the shear strength parameters were obtained using consolidated undrained triaxial tests with pore pressure measurements. The authors concluded, on the basis of analysing nine stable slopes and three landslides, that total stress analyses gave unreliable results and that effective stress analyses according to Bishop (1954), which gave a scatter of the factor of safety of as much as $\pm$ 15% from unity, were suitable for the analysis of the stability conditions of natural slopes

in normally consolidated clays.

2.3 Slopes in Overconsolidated Intact Clays

For the purposes of this section, two natural slopes concerning overconsolidated intact clays which have been analysed in terms of effective stresses are considered. These have been reported by Sevaldson, (1956) and Skempton and Brown (1961) and are presented briefly in the following paragraphs.

The landslide described by Sevaldson occurred in a cutting excavated to a total depth of about 58 ft., at a slope of 2:1 at Lodalen, Oslo, Norway. The slope was comprised of comparatively homogeneous, lightly overconsolidated late glacial and post-glacial clays with some thin layers of silt. In this instance, no estimate of the amount of overburden which caused the overconsolidation was made. The chronology of the deposits was deduced by examination of micro fossils within the strata. The failure occurred six years after some modifications were made to the cutting which had originally been constructed 30 years before the failure.

A comprehensive site investigation was performed consisting of insitu vane tests and "undisturbed" sampling of the soils. Piezometers of the N.G.I open standpipe type were installed mainly outside, but also within the slide area to measure the pore water pressures. The position of the slip surface was located from careful inspection of the "undisturbed" samples and consideration of the geometry of the slide area and was assumed to be circular in shape. Effective stress shear strength parameters were obtained from triaxial tests on the "undisturbed" samples. The true cohesion and true angle of internal friction were determined for some of

the samples using drained triaxial tests and it is significant to note that average values of these parameters did not differ appreciably from those obtained from consolidated undrained triaxial tests with pore pressure measurements.

Three sections through the landslide were analysed using a total stress analysis and two methods of effective stress analysis. The effective stress analyses used were the conventional slices method (May and Brahtz, 1936), and Bishop's method (1954). The total stress analysis gave a factor of safety ranging from 0.85 to 1.15, according to the range of undrained shear strength determined. The May and Brahtz method gave a factor of safety of 0.84, and Bishop's method a factor of safety of 1.05. Sevaldson concluded that in this instance, the total stress analysis would have given a fairly accurate determination of the stability if the shear strength used in the analysis had corresponded to that which could have been measured before failure occurred. He considered that the conventional slices method was overconservative and that Bishop's method gave a more correct estimate of the stability conditions.

The landslide described by Skempton and Brown (1961), occurred in a valley slope of the river Lune near Middleton-in-Teesdale, Yorkshire, England. The slope was 42 ft. high at an inclination of 1.9:1 and was comprised of heavily overconsolidated boulder clay or till. In this instance, the clay was estimated to have been overconsolidated by some 500 to 700 ft. of deposits subsequently removed by erosion. The underlying strata consisted of sandstone, shale and limestone. The site investigation included "undisturbed" sampling of the till and installation of piezometers in both the till and the underlying bedrock. Insufficient

piezometric data were obtained and hence it was necessary to use the upper and lower limits for the groundwater flow pattern. These limits were determined on the basis of the data that was available and the geology of the site. Analyses of the stability were made using these limiting conditions and also assuming horizontal flow, and flow at the surface of the slope.

Peak shear strength parameters were obtained from seven sets of drained triaxial tests and one set of undrained triaxial tests with pore pressure measurements on samples of undisturbed and remoulded till. These tests were generally performed in accordance with the procedures outlined by Bishop and Henkel (1962), using porous plates at both ends of the samples and filter strips around the outside of the samples. For most of the drained tests, the deviator stress was applied at a rate such that failure occurred after not less than 5 hours, corresponding to 98% equalisation of pore pressure as calculated from the formula given by Gibson and Henkel (1954). Two sets of the samples were taken to failure in 2 days and the results were not significantly different from those obtained in the five-hour tests. For one set of the drained tests, five samples were failed in the usual manner by increasing the deviator stress and maintaining the cell pressure constant, and four samples were failed by reducing the cell pressure. A common tangent to the nine Mohr circles was drawn to obtain the shear strength parameters for the set.

It is assumed that the site investigation did not reveal the position of the slip surface since Skempton and Brown analysed the slope using the critical circle through the toe of the slope and assuming a shallow slip plane parallel to the slope. The slope was analysed, in

terms of effective stresses, according to the methods of Bishop (1954), Bishop and Morgenstern (1960), Skempton and Delory (1957) and Henkel and Skempton (1955). A simple total stress analysis (Taylor, 1948) was also performed using undrained strengths obtained from triaxial tests on undisturbed and remoulded samples. The main findings of these analyses were as follows;

(a) Using Bishop's method and Bishop's and Morgenstern's coefficients with peak shear strength parameters, a factor of safety ranging from 0.99 to 1.14 was obtained for the critical circle, depending on the groundwater flow pattern. Allowance was made for a tension crack.

(b) For a factor of safety of unity, Skempton's and Delory's method gave realistic effective stress shear strength parameters assuming a shallow slip plane and groundwater flow at the surface of the slope.

(c) Using Henkel's and Skempton's method assuming zero cohesion and the peak angle of shearing resistance, the factor of safety for the critical circle was 0.65.

(d) The total stress analysis, using Taylor's coefficients with allowance for a tension crack, gave a factor of safety of 2.70.

2.4 Slopes in Overconsolidated Fissured Clays

Many field studies have been made to obtain a method of stability analysis having general applicability to slopes in overconsolidated fissured clays. Generally, total stress analyses have given unreliable results (Bjerrum and Kjaernsli, 1957, Bishop and Bjerrum, 1960). Both in Europe and western Canada, established methods of effective stress analysis which the preceding discussion has shown were suitable for

most other types of clay, have not yeilded satisfactory results for natural slopes in overconsolidated fissured clays. Those landslide studies concerning these soils which appear to be relevant to this research study are those reported by Henkel and Skempton (1955), Henkel (1957), Skempton and Delory (1957), Hardy (1957), Peterson, Jaspar, Rivard and Iverson (1960), Hardy, Brooker and Curtis (1962), Ringheim (1964), and Skempton (1964). They are described briefly in chronological order.

The landslide described by Henkel and Skempton (1955) occurred in the valley of the river Severn, at Jacksfield, Shropshire, England. The slope was about 130 ft. high at an inclination of 5.25:1 (10.5°) and was comprised of stiff fissured clays. Boreholes were put down to recover "undisturbed" samples, measure groundwater levels and locate the slip surface. It appears that no piezometers were used. These boreholes indicated that the failure zone consisted of a 2" thick layer of softened clay and that no artesian pressures contributed to the slide. The appearance of the landslide and the borehole data indicated that the movement had taken place on a relatively shallow plane approximately parallel to the ground surface. A comprehensive series of tests, including drained triaxial and direct shear tests and undrained triaxial tests with some pore pressure measurements, was performed to obtain the shear strength parameters. The clay was classified as an inactive* inorganic clay of low plasticity, and it was discovered that remoulded samples of the soil had slightly higher strength than undisturbed samples. It is of interest to note that in the undrained tests, large negative pore pressures were developed and under small normal loads in the drained tests, large increases in volume took place. The landslide was analysed by an infinite

slope analysis, using two sets of effective stress shear strength parameters, and assuming flow parallel to the ground surface. Additionally, the undrained strengths of the clay from within and outside the failure zone were used for total stress analyses. Using the undrained strength of the clay from outside the failure zone, the total stress analysis gave a factor of safety of 4.0, but using the undrained strength of the clay from the failure zone this value was reduced to 1.12. The effective stress analysis using the peak shear strength parameters gave a factor of safety of 1.45; however, assuming zero cohesion and the peak angle of shearing resistance this figure was reduced to 1.07. This latter method of effective stress analysis is referred to as the "zero-cohesion" analysis for the purposes of this thesis.

Henkel and Skempton concluded that the total stress analysis gave a satisfactory check on the stability in this instance since the undrained strength of the soil from the very thin failure zone was used. However, they pointed out that using the undrained strength of the soil from above or below the failure zone this method gave a ridiculous assessment of the stability. The best estimate of the stability was though to be that obtained using the "zero-cohesion" analysis. They cited the results of earlier laboratory studies (Bishop and Henkel, 1953), and the work of Terzaghi (1936) to support their adoption of this method.

In 1957, Henkel analysed the failures of two cuttings, one at Woodgreen and the other at Northolt, London, England. At both sites, the soil involved was London clay which is a heavily overconsolidated fissured clay. In this instance, the clay has been overconsolidated by 500 to 700 ft. of overburden subsequently removed by erosion. The slope at Woodgreen was supported by a retaining wall and had been excavated,

55 years before its failure, at an inclination of 3:1 to a depth of 37 ft. The slope at Northolt was constructed at an inclination of 2.5:1 and was 33 ft. deep. Few details of the site investigations were given but it is known that borings were made in the slide area at the Woodgreen site, in which 2" pipes were used to measure groundwater levels, and at the Northolt site, Casagrande type piezometers were installed outside the slide area to measure the pore pressures. A wedge analysis, in terms of effective stress was made for the Woodgreen slope, and a slip circle analysis according to Bishop (1954), for the Northolt slope. From these, Henkel concluded that using the "zero-cohesion" analysis, the factors of safety for the two slopes were underestimated by 20%. If peak cohesion and angle of shearing resistance were used, the factors of safety were overestimated by 30%.

In 1957, Skempton and Delory reported further evidence to support adoption of the "zero-cohesion" analysis for natural slopes in overconsolidated fissured clays. By use of a vernier clinometer, they determined the average inclination for limiting stability of a large number of natural slopes in London clay. It was noted that the unstable slopes generally had a water table at or near the ground surface, and using a "zero-cohesion" analysis based on infinite slope theory, they obtained a factor of safety of unity for these observed limiting conditions.

In western Canada, much of the prairie area is underlain by very heavily overconsolidated fissured clays. The main differences between these clays and those found on the European continents is firstly their mineralogy in that these soils contain significant amounts of montmorillonite and secondly, their stress history as they have been consolidated by some 1,500 ft. of sediments and 3,500 to 10,000 ft. of ice. In 1957, Hardy

described landslides in these soils at three locations. These were along the Alaska Highway, near Dawson Creek in British Columbia, at Pat's Creek, near the town of Peace River in Alberta, and at Edmonton, Alberta.

The slope on the Alaska Highway was 120 ft. high at an inclination of about 5:1 (11.25^{o}) and the highway was located at an elevation of about 20 ft. below the crest. Instability developed over a period of 5 years during which precipitation was above normal. Boreholes put down in the slide zone showed that the slope was comprised of hard clay shales with very thin layers of sand and gravel and some softer zones of shale. These were underlain by a sand layer at a depth of about 55 ft. below the slope. Remnants of permafrost were encountered within the slope in one of the holes. Casagrande type piezometers were installed in several of the holes and showed small artesian pressures. A stability analysis of the slope using the Ø circle method with the maximum recorded seepage pressure and average soil characteristics from laboratory tests gave a factor of safety for the slope of about 2.5. An analysis for the stability under "sudden spreading" conditions (Terzaghi and Peck, 1948), assuming that the sand layer at depth was in a quick condition at the time failure occurred, gave a factor of safety of 0.9. Hardy pointed out that although the latter analysis gave a reasonable answer, firstly, there was no indication of sand flow from the toe of the slope and, secondly, the measured pore pressures did not correspond with those required for this type of failure.

The slope at Pat's Creek was 275 ft. high at an inclination of about 3:1, and failed about two years after the construction of a highway across the slope. Boreholes to depths of as much as 200 ft. indicated

very non-uniform soil conditions. The strata were comparatively thin and ranged from silty gravel, through sand, silt, silty sand, silty clay to clay with a wide range of plasticity. Silty sand or thin sand layers yielded free water and springs appeared at isolated locations on the hill-side. Unfortunately, no analyses of the stability of this slope were reported; however, the author expressed objections to the "zero-cohesion" analysis (Henkel and Skempton, 1955) and postulated that a reduction in shear strength could only result due to high seepage pressures. He further postulated that in the Pat's Creek area, very few existing stable slopes would be shown to be stable if analysed using the "zero-cohesion" method.

The third landslide described concerned a section of the bank of the North Saskatchewan River, known as Grierson Hill, in the city of Edmonton. The height of the bank at this location was 170 ft. and 78 years ago it had an inclination of about 2.5:1. Records show that major failures of the bank occurred 60 years ago and 50 years ago, after periods of abnormal precipitation. In the early part of this century, some 50 ft. of random sanitary fill was placed on the lower portion of the slope in an effort to stabilise it and the inclination of the bank was reduced to about 5:1. Although no major failures occurred after placement of the fill, when a highway embankment was constructed along part of the crest of the bank considerable creep movements in the fill resulted. Hardy reported that these movements amounted to as much as 16 ft. per year in the 1950's. Between 1951 and 1953, a comprehensive site investigation showed that the subsoil profile consisted of beds of sand, silt, sandstone, a variety of clay shales and coal. Several coal seams showed free water with some artesian pressures and these were believed to be the cause

of the instability. In 1953, a series of wells were located along the highway embankment and when pumped continuously these prevented major movements of the bank during one of the wettest summers on record. Movement of the lower portions of the slope was not completely arrested by this operation but was reduced to half of its original rate. No stability analyses were reported.

Between 1950 and 1959, extensive investigations were carried out by the Prairie Farm Rehabilitation Administration in connection with several water development projects in western Canada. Several of these investigations were summarised by Peterson et al. (1960).

Peterson et al described construction stability problems at the sites of the Seven Sisters Falls project, Winnipeg, Manitoba and the North Ridge Dam in southern Alberta. Also several natural slopes in the proximity of the South Saskatchewan River Dam were analysed. On the basis of many analyses the authors concluded that;

> "Stability predictions based on laboratory shear strengths have serious limitation for both slightly and heavily preconsolidated clays of high plasticity. This applies to both effective and total stress methods of analysis."

It is considered appropriate to review these case histories in some detail.

Seven Sisters Dikes, Winnipeg, Manitoba

These dikes were constructed in two stages to a total height of about 17 ft. of medium to highly plastic clay with water contents 5 to 10% above the standard Proctor optimum. The upstream slopes were 3.5:1 and the downstream slopes 2.5:1. The first stage was completed in 1931, but during the second stage, in 1949, minor slope failures and settlements occurred.

Four months after construction 13 slides took place in a length of 3.5 miles. Borings showed that the foundation of the dikes consisted of highly plastic clay to depths of 10 to 20 ft. underlain by a clay of low plasticity or bedrock. Core samples from the slide areas were examined but did not disclose the position of the slip surfaces; however, it was concluded that the failures occurred within wetter zones of the foundation soils. Undrained shear strengths of the foundation clays were obtained using field and laboratory unconfined compression tests and a field vane. Some of the laboratory tests were carried out with the fissures in the soil samples oriented parallel to the failure plane and at 45^{o} to the failure plane but these showed no significant variation in strength. Effective shear strength parameters were obtained for the embankment and foundation clays from a very large number of consolidated undrained tests with pore pressure measurements and consolidated drained tests. Stability analyses were performed, in terms of total and effective stresses, for a typical section of the embankment and these gave factors of safety of 1.31 and 1.40, respectively. The other sections of the dike were analysed in terms of total stresses and gave factors of safety ranging from 0.90 to 2.10, the average value being 1.50. Additionally, two slopes which failed during construction were analysed. Using steady seepage pore pressures based on Casagrande's construction of the phreatic line, the factor of safety from an effective stress analysis was 1.40 for a foundation slip circle passing through the downstream slope. The authors reported that engineers from the Norwegian Geotechnical Institute suggested modifications to the pore pressure assumptions and recommended that the phreatic line be considered as horizontal from the

upstream water level due to the possibility of horizontal stratification in the fill. The final stability analyses performed by the authors using the elevated phreatic line gave a factor of safety of 1.33, ignoring the forces on the sides of the slices, and 1.65 if these forces were considered. It would appear significant to note that for the purposes of these analyses, the horizontal phreatic line was considered to be the worst condition of pore pressure within the dikes and construction pore pressures were considered to be of secondary importance.

North Ridge Dam, Southern Alberta

The failure of this embankment was first described by Peterson, Iverson and Rivard in 1957. The total height of the dam was intended to be 70 ft. but movements took place in the foundation when the construction height was 50 ft. The foundation soils consisted of a 10 to 19 ft. layer of highly plastic clay.

Stability analyses were made, in terms of total and effective stresses for the embankment conditions prior to the failure. The best factor of safety for a total stress analysis was 1.20 and 1.06 for an effective stress analysis using estimated pore pressures. The embankment was safely completed in 1956 after sufficient dissipation of pore pressures had been recorded. It is of interest to note that a piezometer installed beneath the final construction stage of the embankment, within the fill, showed a pore pressure increase with construction appreciably greater than most of the piezometers in the foundation.

South Saskatchewan River Dam

The design of this dam was influenced by a study of existing valley slopes in the proximity of the site. These valley slopes showed

both stable and unstable sections, the latter being marked by crescent shaped depressions which are indicative or previous movements. Generally, both types of slope section were at inclinations ranging from 8:1 to 12:1 and were mainly comprised of Bearpaw clay shale. Extensive site investigations indicated that this shale was soft and weathered near the ground surface and increased in strength and density with depth. In the upper weathered zone, there were many slickensides and fissures and there was evidence of previous sliding. There were also a few thin layers of bentonite. In the lower zones, there were fewer fissures and the shale was relatively homogeneous. A pressure test section installed at about river level, 400 ft. from the river edge and 60 ft. below ground level, showed a vertical pressure of 100% overburden and a horizontal pressure, parallel to the river, of 150% of overburden. These test excavations showed that the shale was very impervious except in the upper weathered zone.

The shear strength of the shale was found to vary considerably depending on the zone from which it had been sampled and the size of the sample. Generally, larger samples gave lower strengths due to the weakening effect of the fissures. The test methods included direct shear tests in a cylinderical shear box, consolidation and swell tests, unconfined compression tests and consolidated undrained triaxial tests with pore pressure measurements. An extensive series of creep tests on hard shale samples indicated a creep strength as low as 50% of the value obtained by conventional unconfined compression tests. The shale, which contains a high percentage of the clay mineral montmorillonite, was found to have an affinity for water and extremely high swelling pressures.

Several slopes were analysed to determine their stability conditions in terms of total and effective stresses, and in all cases reported including a "zero-cohesion" analysis, the factors of safety were well above unity. It was concluded that the shear strength of natural slopes was related to the long-time creep strength rather than the short-time shear strength as obtained in normal laboratory tests, and the creep strength was adopted for design purposes.

In 1962, Hardy, Brooker and Curtis described two failures which occurred in northern Alberta. These were located at Dunvegan Creek, between Grande Prairie and Peace River and at the Little Smoky River between Valleyview and Falher. The Dunvegan slide occurred in a valley slope about 700 ft. high at an inclination of about 9^{o}, during the construction of a highway embankment. Some 70 ft. of the embankment was constructed in the fall of 1958. The failure occurred about 2 weeks after the construction was continued in the Spring of 1959, and involved an area of about 50 acres of the valley side. Boreholes, put down before the highway construction, showed medium to highly plastic dark brown and grey inorganic clays interbedded with silt and silty sand seams overlying a soft sandstone bedrock. Free water was located in most of the holes but no significant artesian water pressures were recorded. After the landslide, many additional holes were drilled. Casagrande piezometers were installed within and adjacent to the slide area where soil tests had shown an increase in the liquidity index of the profile soils. None of the piezometers in the clay showed measurable pore pressures but small artesian pore pressures were recorded in the silty sand. The slip surface was located by means of probewells for which 2" galvanised pipe and 3/4" plastic tubing

were used. The ratio of the length of the slide to its depth was found to be about 20:1.

The effective shear strength parameters were obtained using consolidated undrained triaxial tests with pore pressure measurements. In addition, many unconfined compression tests were performed. Effective stress stability analyses based on infinite slope theory gave the following results:

(a) Using peak shear strength parameters, and zero pore pressures, a factor of safety of 2.81 was obtained.

(b) Using a "zero-cohesion" analysis with zero pore pressure, the factor of safety was 2.50.

(c) Using a "zero-cohesion" analysis with the maximum recorded value of pore pressure, the factor of safety was 1.77.

A total stress analysis, using a method of slices, gave a factor of safety of 2.60.

The Little Smoky River slide involved movements of a bridge pier founded on piles driven into clay shales within the river bank. The slope at the bridge was 300 ft. high and had an inclination of about 12°. The stratigraphy and pore pressure observations at this site were similar to those found at the Dunvegan site, and similar methods were adopted for the stability analyses. In all cases, the factors of safety were well above unity.

The authors suggested that Coulomb's equation be modified by subtracting the swelling pressure of the soil from the total stress to obtain an effective stress. Their procedure may be illustrated by the introduction of the term "P_s" in the following equation;

$$\tau = c' + (\sigma - u - P_s) \tan \phi'$$

where: τ = effective shear strength

c' = effective cohesion

σ = total normal stress

u = pore pressure

P_s = swelling pressure from constant volume swell test

ϕ' = effective angle of shearing resistance.

Using this modified equation, factors of safety of 0.97 and 0.98 were obtained for the Dunvegan and Little Smoky landslides, respectively. The swelling pressures used in the equation were determined from constant-volume swell tests performed in a one-dimensional consolidation apparatus with distilled water as the immersing fluid. More recent studies indicate that by using distilled water as the immersing fluid higher swelling pressures are caused than those able to occur in nature where the soil pore fluid would contain appreciable dissolved salts.

Ringheim (1964) described field observations on the performance of the Bearpaw Formation during the construction of the South Saskatchewan River Dam. During construction, many Wilson type slope indicators were installed in excavation and embankment foundation areas and often these indicated movements within the fill although no surface movements were observed. These movements occurred in the soft shale zone of the Formation when the construction had reached about half the height of the dam. Some of them involved very large quantities of the embankment fill and, at several locations, the plane of movement at the toe was in a layer

of bentonitic clay shale.[1]

U.S.B.R. and Casagrande type piezometers were installed in the shale foundation and observations were made as the construction took place. The Casagrande type piezometers showed a time lag in pore pressure change with loading or unloading. Also, when the amount of overburden removed or replaced was less than 50 ft., only 50% of the overburden was recorded in pore pressure change. Over 50 ft., 75% of the overburden was recorded in pore pressure change. Unfortunately, no similar observations were reported for the U.S.B.R. type piezometers.

Analyses of the construction failures were made using Bishop's method (1954) for slip circle geometry and Janbu's method (1956), for sliding block geometry. Assuming a factor of safety of unity, the effective stress shear strength parameters for the foundation clay shales were then obtained. These were used for the re-design of the embankment slopes and were reported as $c' = 0$ and $\phi' = 9^{\circ}$.

In 1964, Skempton re-examined the concepts of "long-term" stability analysis of natural slopes. He stated;

> "From the analysis of actual slips in clays, the values of the shear strength parameters, as determined by conventional tests, do not necessarily bear any relation to the values which must have been operative in the clay at the time of failure. This conclusion which has now been established beyond the slightest doubt, is obviously one of immense practical significance. We must therefore endeavour to understand why, in certain cases, there is a wide discrepancy between our ordinary laboratory test results and the actual values of shear strength".

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1. The P.F.R.A. have not reported any shear strength test results for this soil to the present time.

Skempton postulated that fissures and joints acted as stress concentrators and discontinuous planes of weakness and were hence a controlling factor in the progressive failure of natural slopes. He pointed out that previous effective stress stability analyses using peak shear strength parameters had given excessively high factors of safety and suggested that instead, such analyses be performed using residual shear strength* parameters. He postulated that the proportion of the slip surface over which the residual strength was mobilised depended upon the gross structure of the clay. He defined the residual factor as the proportion of the total slip surface over which the residual strength was operative at the time of failure.

Three of the landslides already described in this chapter were re-analysed by Skempton using residual shear strength parameters determined from direct shear tests carried out to large strains. The results of these analyses were as follows;

(a) The slope in overconsolidated intact clay, analysed by Skempton and Brown (1961), had a factor of safety of 0.69, and a residual factor of 0.08.

(b) The Jacksfield slope (Henkel and Skempton, 1955) had a factor of safety of 1.11 and a residual factor of 1.0.

(c) The Northolt slope (Henkel, 1957) in London clay, had a factor of safety of 1.0 and a residual factor of 0.56.

2.5 Summary of the Review

The preceding case histories concern stability studies which are relevant to the present research. The following sub paragraphs summarise

the important findings of the review in relation to the investigation and analysis of the LeSueur Landslide and the existing concepts of slope stability analysis.

2.5 (a) Site Investigations

Relatively few site investigations have included careful measurement of the position of the slip surface and in several instances pore pressures have been assumed rather than measured. Since both factors have a profound influence on the results of an analysis, such results must be considered to contain some unknown degree of error. In many cases, slip surfaces have been assumed circular in shape for the purposes of stability analysis.

2.5 (b) Laboratory Testing

Much attention has been given to performing shear strength tests to define the peak shear strength parameters of the soils involved in the failures. Generally, the peak shear strength parameters have not varied significantly when a large variety of tests have been performed on the same soil. The parameters used in the case histories considered were determined using standard laboratory procedures including direct shear tests, consolidated undrained triaxial tests with pore pressure measurements and drained tests. In effect, all test types were employed.

2.5 (c) Selection of Parameters for use in analysis

Total Stress Parameters

Total stress analyses, using the undrained shear strength from unconfined compression tests or undrained triaxial tests are not generally

valid for estimating the stability conditions of any natural clay slope.

Peak Effective Stress Parameters

It would appear that the effective stress analysis, using peak shear strength parameters and measured field pore water pressures, is suitable for the analysis of natural slopes in normally consolidated and overconsolidated intact clays. (Skempton, 1945, Bjerrum and Kjaernsli, 1957, Sevaldson, 1956, Skempton and Brown, 1961). However, it should be pointed out that in several instances the factors of safety were in error by as much as 15%.

Modified Effective Stress Parameters

Three modifications to the effective stress parameters have been used for assessing the stability conditions of natural slopes in overconsolidated fissured clays. These were reported by Henkel and Skempton (1955), Hardy, Brooker and Curtis (1962) and Skempton (1964). Respectively, these modifications concern the "zero-cohesion" concept, the introduction of the swelling pressure, and the use of the residual strength.

2.5 (d) Factors concerning the Present Study

Several factors arising from the review were considered to have a bearing on the conduct of the present study. Measures were adopted, in accordance with these factors, to minimise the assumptions concerning the field conditions and to obtain as much pertinent laboratory testing data as possible in the time available.

1. Since the results of stability analyses based on assumed slip surfaces

and pore pressures contain some unknown degree of error an attempt was made to minimise these assumptions for the LeSueur Landslide analysis. Probewells were installed within the slide area to ascertain the position and shape of the slip surface and hydraulic type piezometers were installed in a stable slope adjacent to the landslide to measure pore water pressure fluctuations.

2. The laboratory procedures employed in the case histories varied over a wide range of test types and techniques, hence, it would appear that any standard test procedure will yield satisfactory shear strength parameters if properly conducted. For the purposes of this investigation, consolidated undrained triaxial tests with pore pressure measurements were used in which the cell pressure was maintained constant and the deviator stress was increased to failure. The rate of strain for these tests was chosen as 1.25% per hour, largely from the point of view of time available to the author.

3. Although analyses based on total stresses were generally considered to be inapplicable to "long-term" stability problems, two such analyses were performed in the present study as they were considered to be of some interest.

4. Generally, the analyses described used peak shear strength parameters with respect to effective stresses. These parameters and the modified parameters according to Henkel and Skempton (1955) were used for the present study. The modified Coulomb equation according to Hardy, Brooker and Curtis (1962) was not used for any analysis as the test procedure necessary to obtain the operative swelling pressures appears to require further research. Due largely to lack of time, residual shear strength

parameters (Skempton, 1964) were not determined; however, based on the assumption that remoulded samples yield strength parameters approaching those obtained from residual strength tests, a series of triaxial tests was performed on a set of remoulded samples.

2.5 (e) Factors concerning the Concepts of Slope Stability Analysis

1. It would appear that the effective stress analysis, using peak shear strength parameters and measured field pore water pressures, is satisfactory for the analysis of natural slopes in normally consolidated clays and overconsolidated intact clays although, in some instances, factors of safety have been as much as 15% in error. In the light of the studies made by Bishop (1957) the author submits that the opinions of Peterson et al. may be somewhat unjustified in regard to slightly overconsolidated clays. It would appear possible that failures of the Seven Sisters dikes may have been caused by construction pore pressures. The authors reported that the fill was placed at a moisture content 5 to 10% above the standard Proctor optimum. From studies performed some time after the authors conducted their stability analyses, Bishop showed that large pore pressures can be built up during the construction of an embankment when soil is placed only a few percent wet of optimum moisture content if drainage facilities are not provided within or below the embankment. Since the foundation of the dikes had a relatively low permeability, little, if any pore pressure dissipation would have been possible during construction. On impounding water, the pore pressures could have increased within the embankments due to seepage and consequently failure may have taken place within the fills rather than in their foundations. Furthermore, the

authors have reported a factor of safety of 1.06 for an effective stress analysis of the north Ridge Dam construction failure which is well within the limits of accuracy previously reported for such stability analyses.

2. Three modifications to the effective stress shear strength parameters have been used for natural slopes in overconsolidated fissured clays each of which has been based on a relatively small number of investigations. Since some of these investigations did not include site measurements of pore pressures or slip surfaces the pertinent stability analyses must contain some unknown degree of error and the worth of introducing modifications to these analyses would appear to be questionable. The author submits that perhaps insufficient attention has been given to the measurement of pore water pressures in overconsolidated fissured clays. The insitu permeability of such clays is undoubtedly highly variable. If these clays have a relatively high permeability (say about 10^{-9} to 10^{-8} cms/sec or more) then it is reasonable to assume that an open standpipe piezometer or hydraulic type piezometer is perfectly suitable for pore pressure measurements. However, under the same conditions of permeability, it is considered unlikely that 2" perforated pipes would ever give a true indication of the pore pressures. If the clays have a low permeability (say less than 10^{-9} cms/sec), then lower pore pressures than actually exist in nature may be measured using a standpipe or hydraulic piezometer due to the measurement time lag. To the present time, field studies of the performance of piezometers in natural strata are few in number; however, two studies have been reported which appear to confirm the significance of the foregoing discussion. Ringheim (1964) has described the performance of double-lead Casagrande piezometers (hydraulic type) used in the Bearpaw clay shale

foundation of the South Saskatchewan River Dam where the measured insitu permeability was as low as 10^{-10} cms/sec. Some of these piezometers indicated a change in pore pressure of only 50 to 75% of the overburden weight added or removed during construction. In contrast, Bishop, Kennard and Penman (1960) using similar instruments at the Selset Dam in England, placed in till having a permeability of 10^{-8} cms/sec, recorded pore pressures of 90 to 100% of the added overburden weight. These differences in pore pressure measurements were probably due to the differences in the permeabilities of the soils concerned, and may indicate that in the case of the Bearpaw clay shale the instruments had an excessive time lag.

If a piezometer having an excessive time-lag under the given soil conditions is used, then lower pore pressures than exist in nature may be measured. When such pore pressures are used for a stability analysis in terms of effective stress then a factor of safety on the high side of unity will be obtained. It is the author's opinion that incorrect or inadequate piezometric studies of natural slopes in overconsolidated fissured clays may well have been an important contributory factor to the existing beliefs that there are wide discrepancies between laboratory test results and actual values of the shear strength of such soils.

CHAPTER III

SITE INVESTIGATION

3.1 Geology of the Area

The geological descriptions given in this chapter are according to Bayrock and Hughes (1962).

The Edmonton area was glaciated during Wisconsin time and the surficial deposits are unevenly distributed. The North Saskatchewan River, which flows in a northeasterly direction through the city of Edmonton, is the main drainage channel of the area. See FIGURE 1. Northeast of Clover Bar, the river occupies a pre-glacial valley and in the proximity of the LeSueur Landslide the geological sequence is as follows;

4. Glacial Lake Edmonton Deposits
3. Glacial Till
2. Saskatchewan Sands and Gravels
1. Edmonton Formation

The Edmonton Formation consists of interbedded sandstones, clay shales and coal seams and is of Upper Cretaceous age. The shales and sandstones are often bentonitic and contain high proportions of the clay mineral montmorillonite. The shales are usually highly fissured. The coal seams are of variable quality and generally only a few inches in thickness.

The Saskatchewan sands and gravels are sporadically distributed throughout the Edmonton district. The sands grade from fine to coarse and

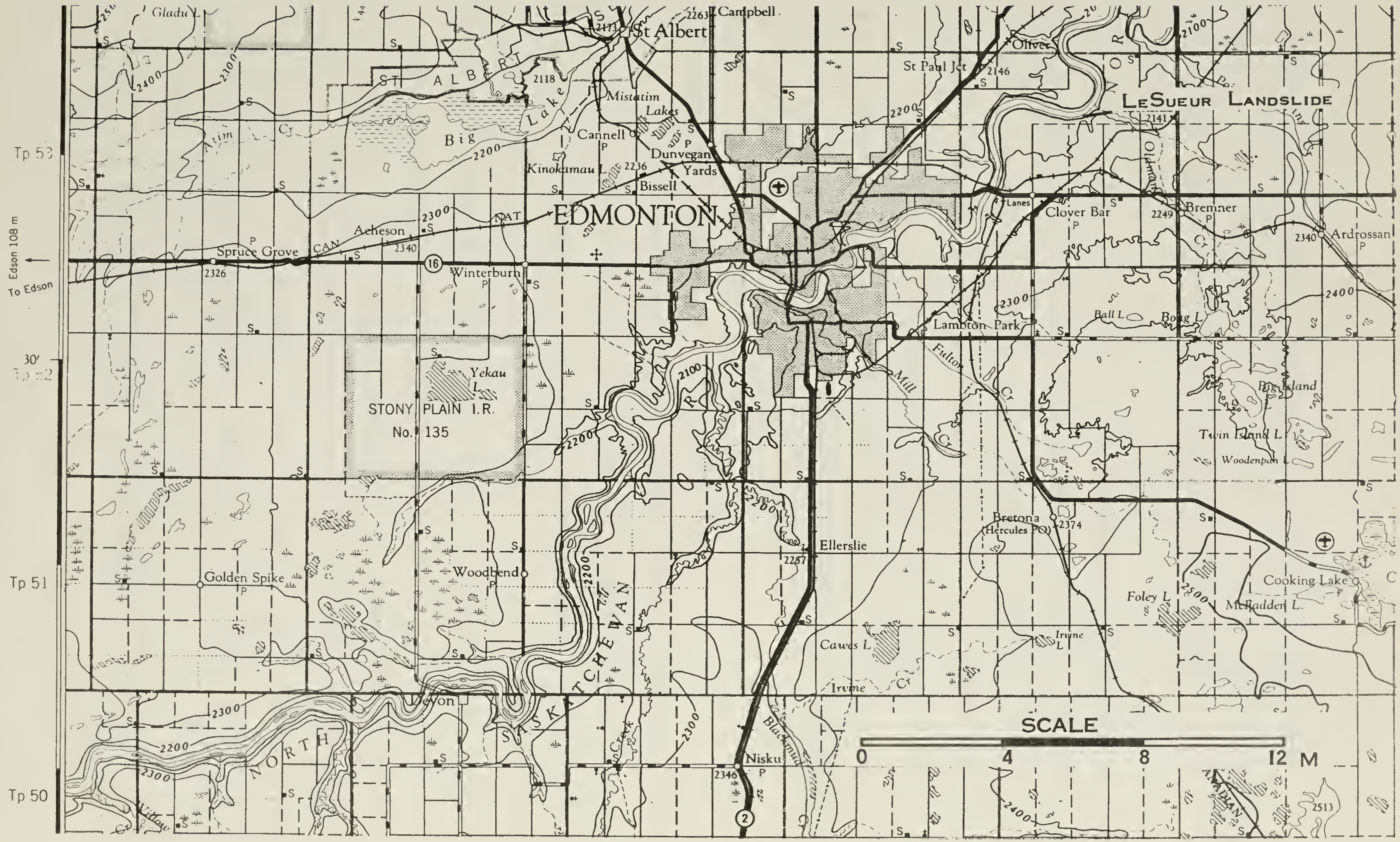

FIGURE I. LOCATION MAP.

the gravels are composed of quartzites and cherts with some arkose pebbles. Since they contain no igneous or metamorphic rocks it has been deduced that these deposits were derived from the Rocky Mountains in pre-glacial times.

The glacial till which contains appreciable quantities of igneous and metamorphic rocks derived from the Canadian Shield, is generally composed of about 40% sand, 30% silt and 30% clay. The clay fraction usually contains a large percentage of montmorillonite. It is a hard cohesive deposit.

The Lake Edmonton deposits were laid down during the period of glacial recession in a preglacial lake which covered a major part of the Edmonton area extending far to the west. The deposits are bedded fine sands, silts and clays.

3.2 Areal Observations

Many landslides take place along the North Saskatchewan River valley in the Edmonton area. The largest of these are located on the outside bank of meander bends of the river. A part of a typical meander bend where landslides have been active for many years is shown in PLATE 1. The LeSueur Landslide, indicated in PLATE 1, was initiated in January 1963, when air temperatures were extremely low, and major movements took place in August 1963. The landslide immediately to the east of the LeSueur Landslide took place in July 1964. It is not known when this landslide was initiated but its geometry and failure pattern are quite similar to those of the LeSueur Landslide. Between these two landslide areas there is a deep "V" notch valley occupied by a small stream. To the west of the LeSueur slide, the high ground curves southward away from the river.

PLATE I. AERIAL VIEW OF LANDSLIDE AREA.

3.3 Preliminary Investigation

3.3 (a) Site Observations

The author first visited the LeSueur Landslide in October 1963, with Dr. S. Thomson, Associate Professor of Civil Engineering, University of Alberta who, later that month, arranged visits with Dr. R.S. Taylor, consulting geologist, Spence Taylor and Associates, Edmonton and Dr. L.A. Bayrock, Pleistocene geologist, Research Council of Alberta, Edmonton.

Many of the strata comprising the river bank were exposed by the slide movements and these were closely inspected. About 30 ft. of the slip surface were seen at the crown scarp and the soils were Lake Edmonton deposits consisting of silts and silty sands with clay inclusions. A view of the crown scarp is shown in PLATE 2. Approximately 10 ft. of vertical movement occurred on the east flank of the slide area. Here, the strata show that the valley side has been slumping for many years at least over a period of two centuries. Several slip surfaces are visible and some of the strata are not in their correct positions in the geological sequence. The Edmonton Formation overlies the Saskatchewan Sands and Gravels and is itself overlain by Glacial till. See PLATE 3. At the lower part of the east flank, a 2 ft. thick layer of bentonite is exposed underlain by a thin weathered coal seam.

At the river edge the strata exposed to the east and west of the slide area are carbonaceous shales and coal and these appear to occupy their original positions in the preglacial valley. However, these strata are not visible in the slide area. Evidently, they have either been pushed outwards into the river by the slide or have been overridden by the toe material. Some decayed timber removed from a slump block at the toe of the

PLATE 2. VIEW OF CROWN SCARP SEPTEMBER 1963.

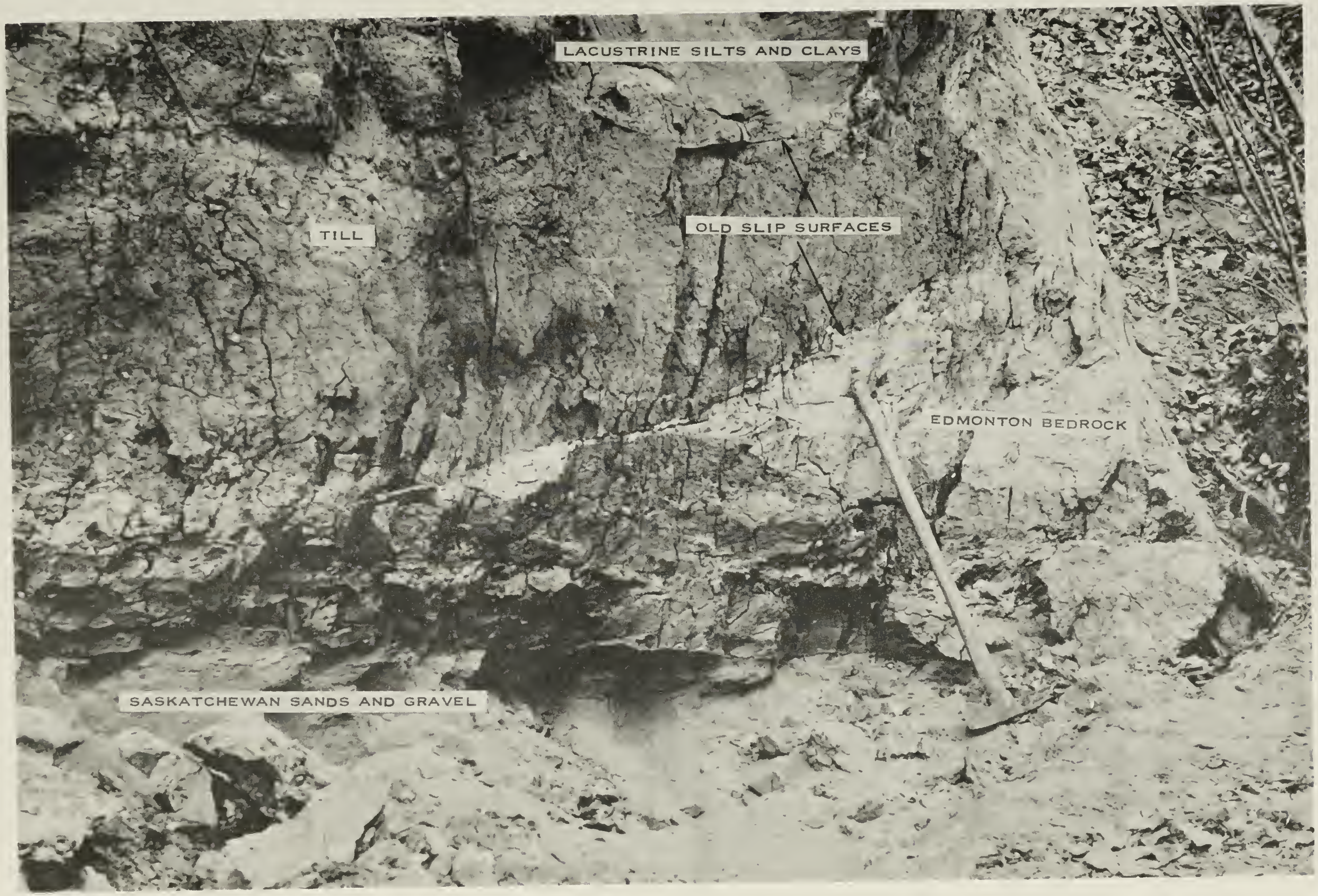

PLATE 3. STRATA EXPOSED ON EAST FLANK.

slide, to ascertain the approximate age of the slump block, was found by Carbon 14 dating to be 170 years $\pm$ 70 years.[1]

The slopes are heavily timbered largely with poplar and spruce. Some of these trees have base diameters upwards of 12". Also, many of the trees including the large ones show distinct curvature in their trunks which is strongly indicative of previous landslide activity. During the major slide movement of August 31st., 1963, it was reported that tree roots were clearly heard snapping and it is quite evident that in a slide of this magnitude tree growth has negligible stabilizing influence.

3.3 (b) Subsurface Exploration

In November 1963, a preliminary subsurface exploration was carried out using funds made available by the Civil Engineering Department of the University of Alberta. Boreholes 1 and 2 were put down, at the positions shown in FIGURE 2, to depths of 119 ft. and 116 ft. respectively, using a truck-mounted rotary drill. These boreholes were drilled dry to a depth of 50 ft. and 3" Shelby tubes were driven where possible to obtain "undisturbed" samples. Below this depth, water-flush drilling was used but due to the hard nature of the soils encountered no suitable Shelby samples were recovered. The boring logs for these holes, which are included in FIGURE 3, showed that a more detailed subsurface exploration would be required to obtain a complete profile of the soils in the river bank. It was realised that continuous coring would be required to recover suitable samples of the Edmonton Formation and that further details of

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1. The Carbon-14 dating was very kindly carried out by Prof. K.J. McCallum, Professor of Chemistry, University of Saskatchewan.

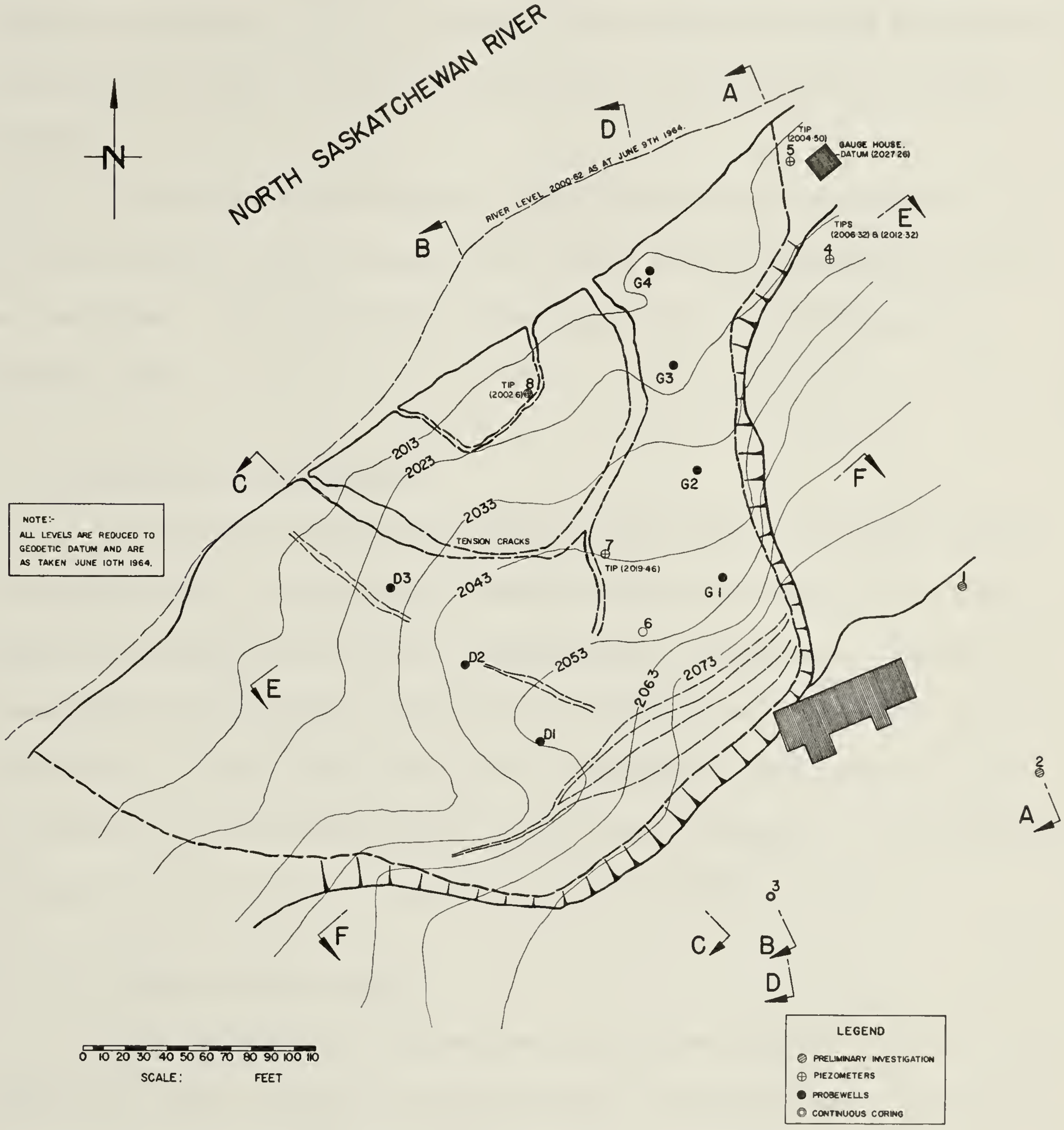

FIGURE 2. PLAN OF LANDSLIDE
LAYOUT OF BOREHOLES & INSTRUMENTATION.

W. T. PAINTER AUG. 64.

the groundwater conditions would be required for the purposes of stability analysis. It was considered that probewells would be necessary to accurately locate the slip surface due to the non-uniform soil conditions.

Disturbed samples of soil were obtained from various parts of the landslide for classification tests and a brief topographical survey was performed. The preliminary investigation was concluded early in December 1963.

3.4 Subsequent Investigation

The LeSueur Landslide study was continued in June 1964, under the sponsorship of the National Research Council of Canada. The subsequent investigation included a comprehensive topographical survey, a more detailed subsurface exploration, probewell and piezometer installations. A site plan, giving the topography of the area and layout of boreholes and instrumentation, is included in FIGURE 2. Stratigraphic profiles through the river bank are given in FIGURES 3, 4 and 5.

3.4 (a) Topographical Survey

The topographical survey was carried out between June 1st. and June 10th., 1964. All major tension cracks were mapped and the levels were reduced to Geodetic datum.[1] The survey included the landslide area and an adjacent stable section of the river bank to the east of this area.

From a topographical plan it was possible to select suitable access routes for the drilling equipment. These were constructed using

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1. Messrs. Hamilton-Olsens precise B.M. No. 659 at the intersection of the CNR main Winnipeg line with the eastern boundary of Section 15 was used for this purpose.

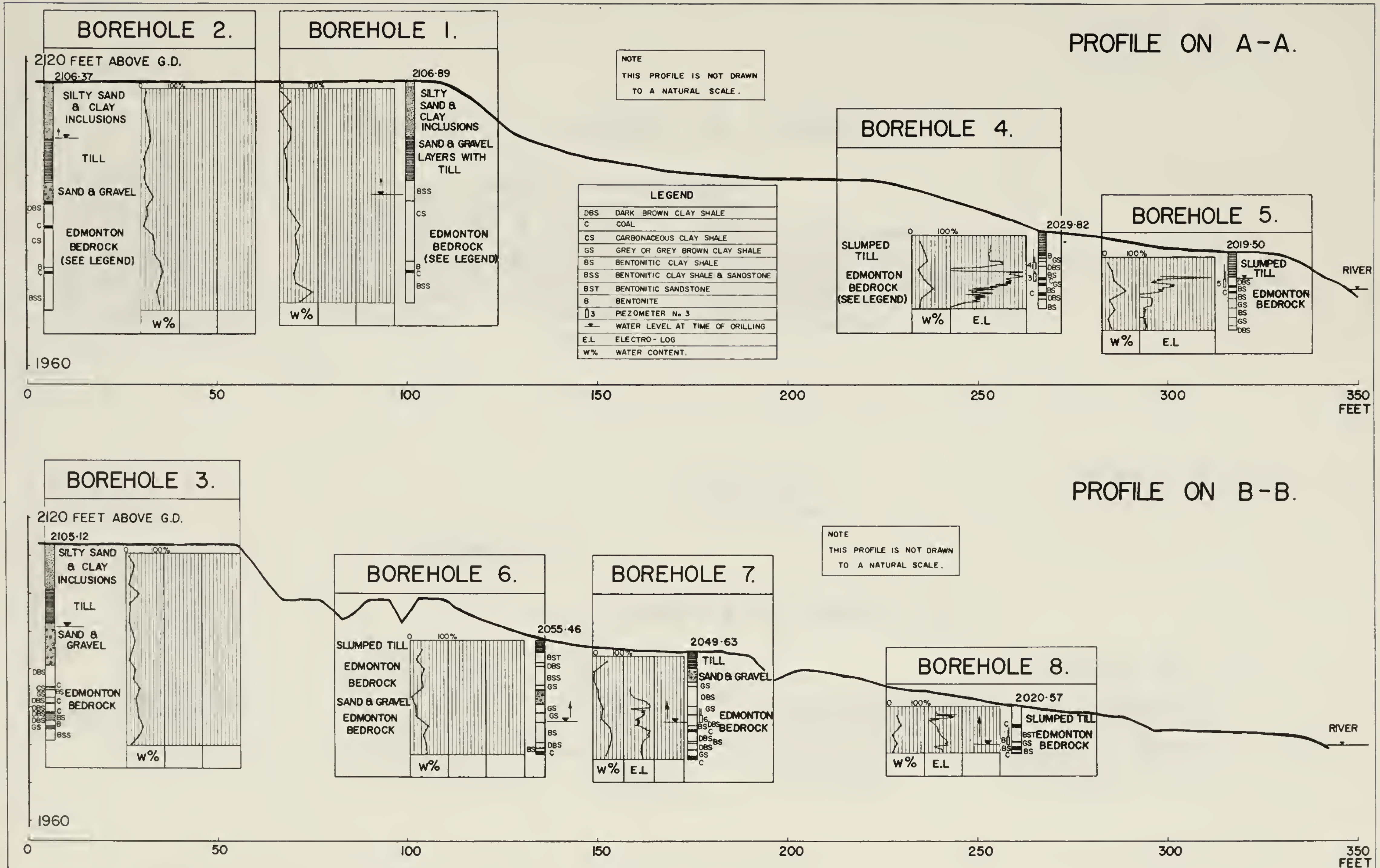

FIGURE 3. STRATIGRAPHIC PROFILES THROUGH LANDSLIDE.

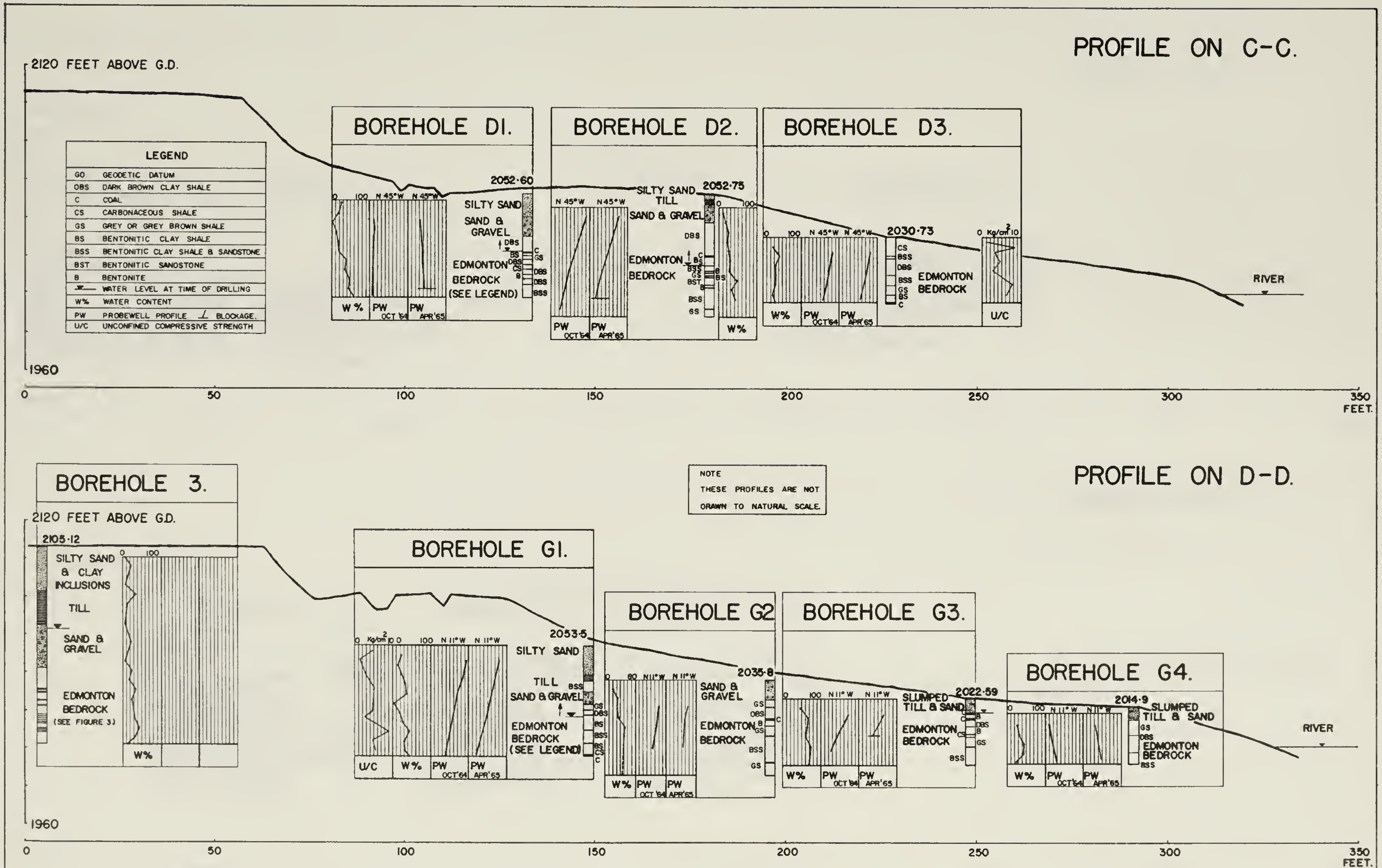

FIGURE 4. STRATIGRAPHIC PROFILES THROUGH LANDSLIDE.

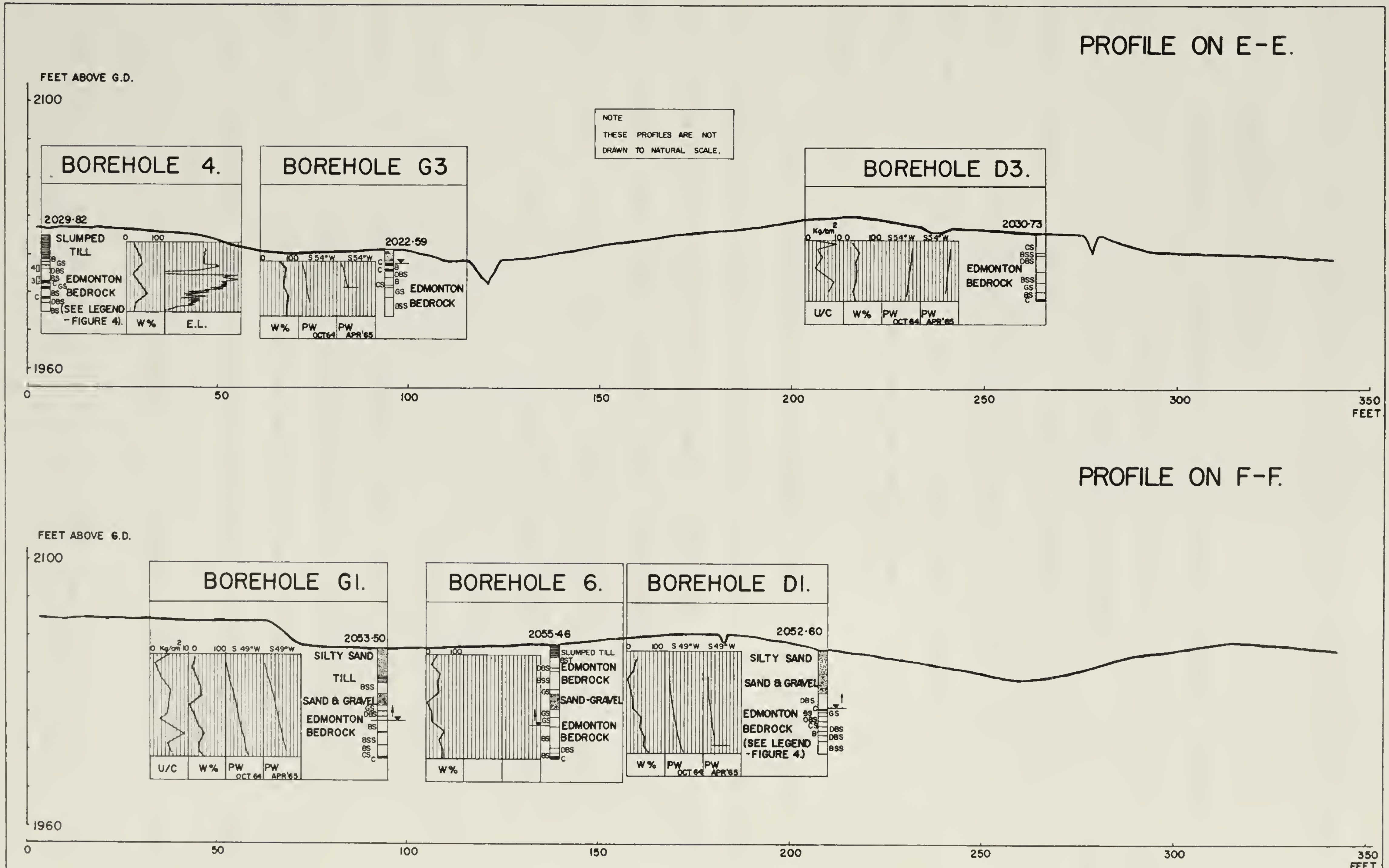

FIGURE 5. STRATIGRAPHIC PROFILES THROUGH LANDSLIDE.

W.T. PAINTER. APRIL 65.

a D8 bulldozer, care being taken to minimise disturbance of the bank.

3.4 (b) Subsurface Exploration

In August and September 1964, thirteen boreholes numbered 3 to 8, G1 to G4 and D1 to D3, inclusive, were drilled at the positions shown on the site plan, to the depths indicated in FIGURES 3, 4 and 5. Several drilling techniques were used. The first boreholes drilled were G2, G3 and G4, using a Bombardier-mounted Failing CSD 2 rotary drill. These holes were intended for probewells and undisturbed sampling was not attempted; however, disturbed samples were taken at 5 ft. intervals to determine the soil profile.

Dry-drilling methods were used for the other 10 boreholes so that uncontaminated samples of groundwater could be obtained and groundwater levels could be accurately located. Borehole 8 was air-drilled[1] for its entire depth using a Boyles BBS 2 truck-mounted rotary drill with the core barrel shown in PLATE 4.[2] This core barrel gave good 4 1/2" diameter core samples of the clay shales using a tungsten carbide bit with an "N" rod drilling stem, but was not successfully used in the soft sandstones. Using the same rotary drill, borehole 3 was air-drilled using a tricone bit to 63 ft. and disturbed samples were taken at 2 1/2 ft. intervals. Below this depth, continuous cores were obtained using an "NMF" core barrel with a tungsten carbide face-discharge bit and mud-flush drilling. Excellent cores were obtained throughout the Edmonton Formation

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1. A 350 cu. ft/min air compressor was located at the top of the bank for this purpose.

2. This core barrel was designed by Mr. J.D. Campbell, President and General Manager, Boyles Bros. Drilling Co. Ltd., Vancouver.

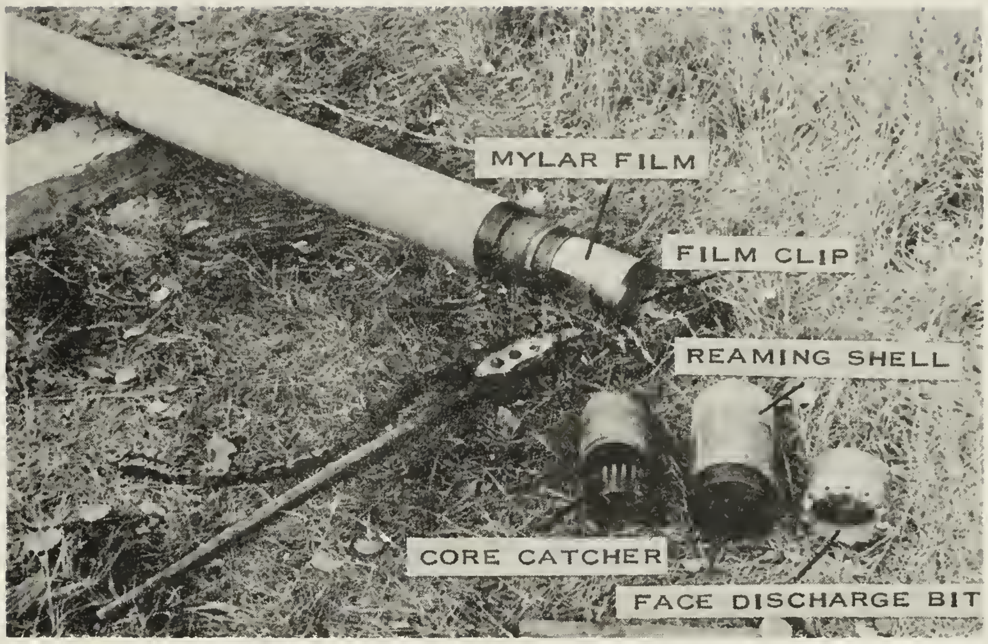


PLATE 4. SPECIAL CORE BARREL.

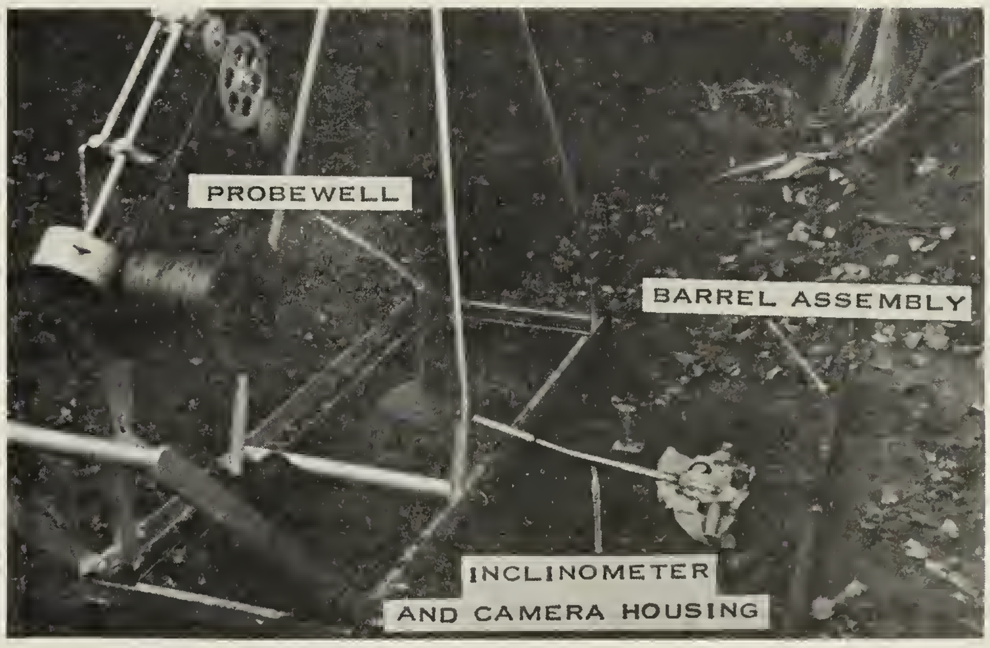


PLATE 5. PROBEWELL INCLINOMETER.

in borehole 3 from 63 to 103 ft. depth. (Elevation 2040.1 to 2000.1 ft. above G.D.).

Boreholes 4, 5, 6, 7, 8, G1, D1, D2, and D3 were dry-drilled using a Bombardier-mounted Mobile B40 drill with 4 1/2" flighted augers. Driven Shelby samples were obtained as possible and disturbed samples were taken at 2 1/2 ft. intervals. Groundwater was located in all dry-drilled holes except D3, and in boreholes 6, 7, 8, D1 and D2 it was struck at depths of 43 ft., 37 ft., 20 ft., 30 ft., and 38 ft. respectively. Groundwater samples were recovered using a simple baling tool.

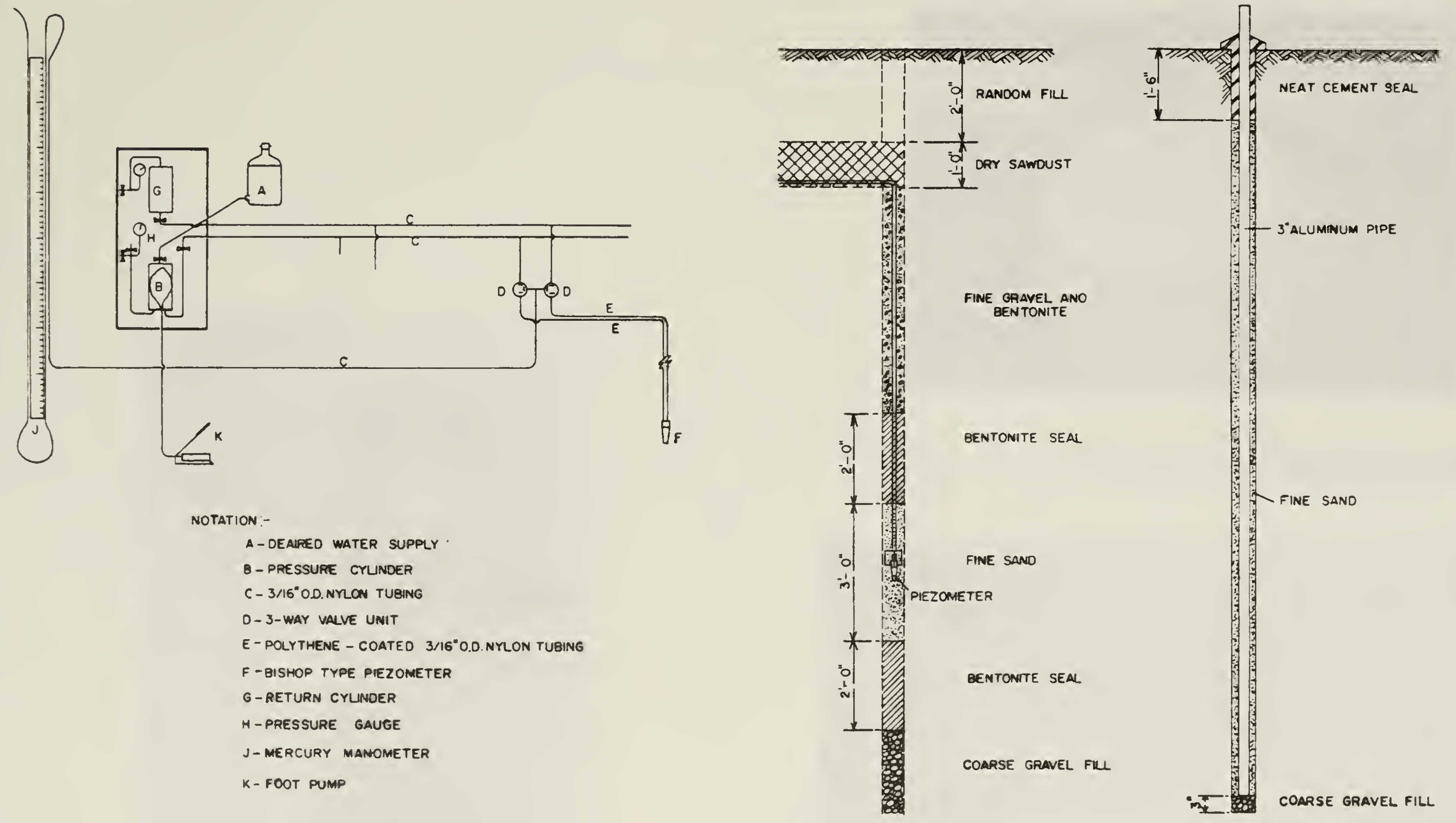
3.4 (c) Piezometer Installations

Five Bishop hydraulic-type piezometers were installed using a light tripod. These were fitted with guide baskets to facilitate their installation in boreholes. See PLATE 6. Two of these were located in borehole 4 and one in each of boreholes 5, 7 and 8. The piezometer tips were located in zones of the profile soils having a relatively high moisture content at the elevations shown in FIGURE 2. These zones were determined using electro-logging similar to a technique reported by Hutchinson (1961). Since this method can only be employed in water-filled open boreholes and some of the boreholes were only part-filled with groundwater the logs are not generally available for the entire depths of the boreholes. See FIGURE 3.

After electro-logging, these four boreholes were cased with 4 1/2" diameter oil-well pipe. This pipe was jacked out of the boreholes as the piezometer installations proceeded. Each borehole was back-filled as necessary with coarse gravel. Clean fine sand was used for the filters and commercial bentonitic drilling mud in small bags was used for the

seals. These bags were made from old nylon stockings which were filled with dry bentonite after they had been soaked in water. Above the upper bentonite seal, the holes were backfilled with a 50-50 mixture of fine gravel and bentonite. All materials were rammed in 6" layers with a modified Casagrande type piezometer hammer which was based on a design used successfully by the P.F.R.A. at the South Saskatchewan River Dam. A typical borehole installation and a simplified diagram of the piezometer system are included in FIGURE 6.

A gauge house was located as near to the toe of the river bank as practicable and was fitted with a de-airing apparatus. This apparatus has been described by Penman (1957). Mercury manometers were used for pore water pressure measurements. Polythene-coated 3/16" nylon tubing was used for the connecting lines from the gauge house to the tips. These lines were placed 3 ft. below ground level in 6" wide trenches excavated with a Davis T78 trencher and were covered with a 12" layer of dry sawdust before the trenches were backfilled. Groundwater was used in the hydraulic system to minimise possible osmotic effects at the piezometer tips. This water was de-aired by passing it through a small diameter coiled polythene tube from the top of the bank to the gauge house. Details of the gauge house instrumentation are given in PLATES 7 and 8. A small propane heater was installed in the gauge house and the system functioned satisfactorily throughout the winter months. Piezometric measurements taken November 1st., 1964, to mid April 1965, have been included in FIGURE 7. The results of the piezometric study are discussed in CHAPTER V.



SIMPLIFIED DIAGRAM OF PIEZOMETER SYSTEM. TYPICAL PIEZOMETER HOLE. TYPICAL PROBEWELL.

FIGURE 6. DETAILS OF INSTRUMENTATION.

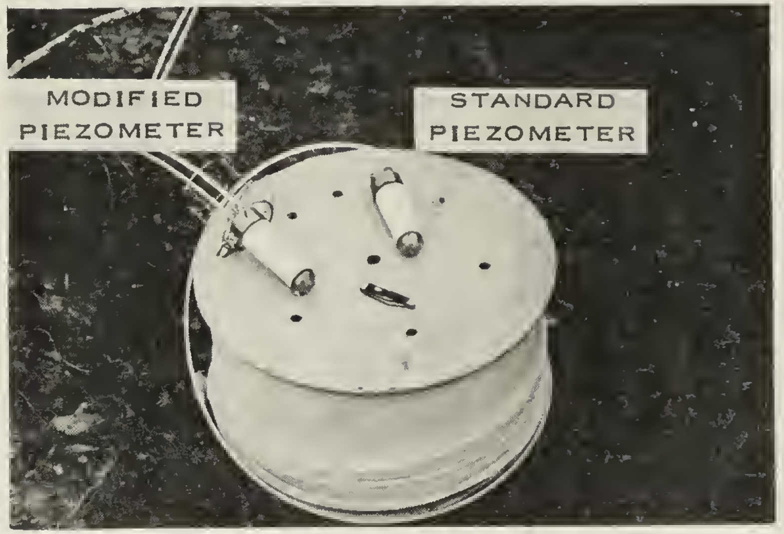


PLATE 6. PIEZOMETER MODIFICATIONS.

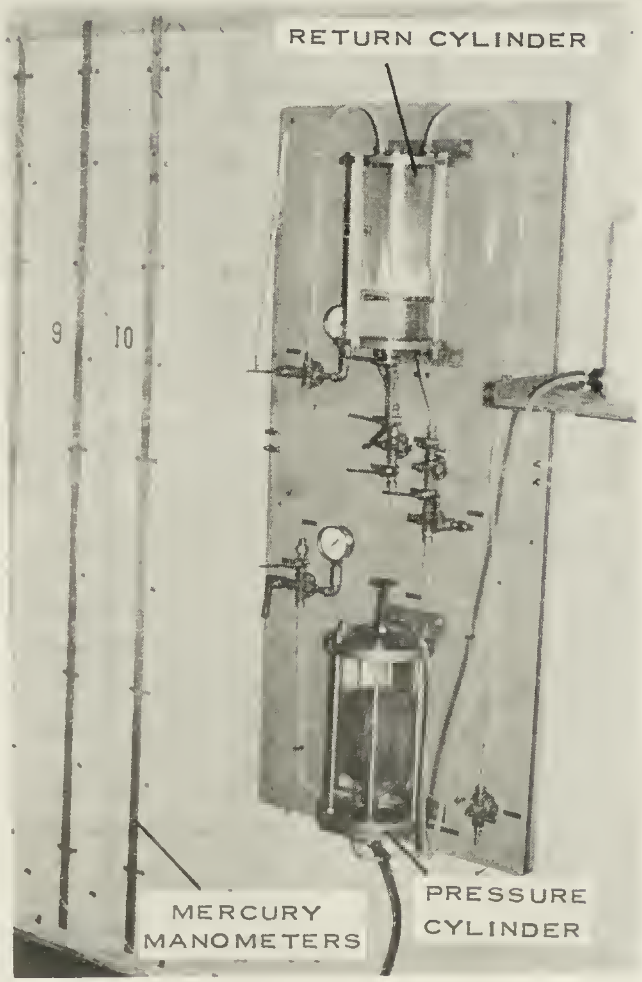


PLATE 7. DEAIRING PANEL.

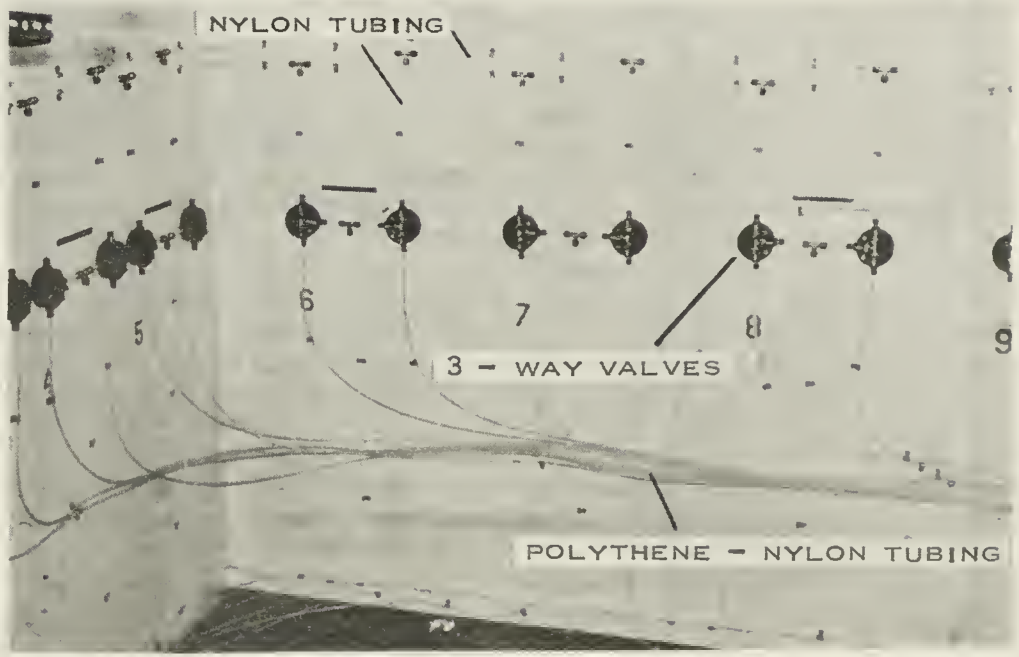


PLATE 8. GAUGE HOUSE PLUMBING.

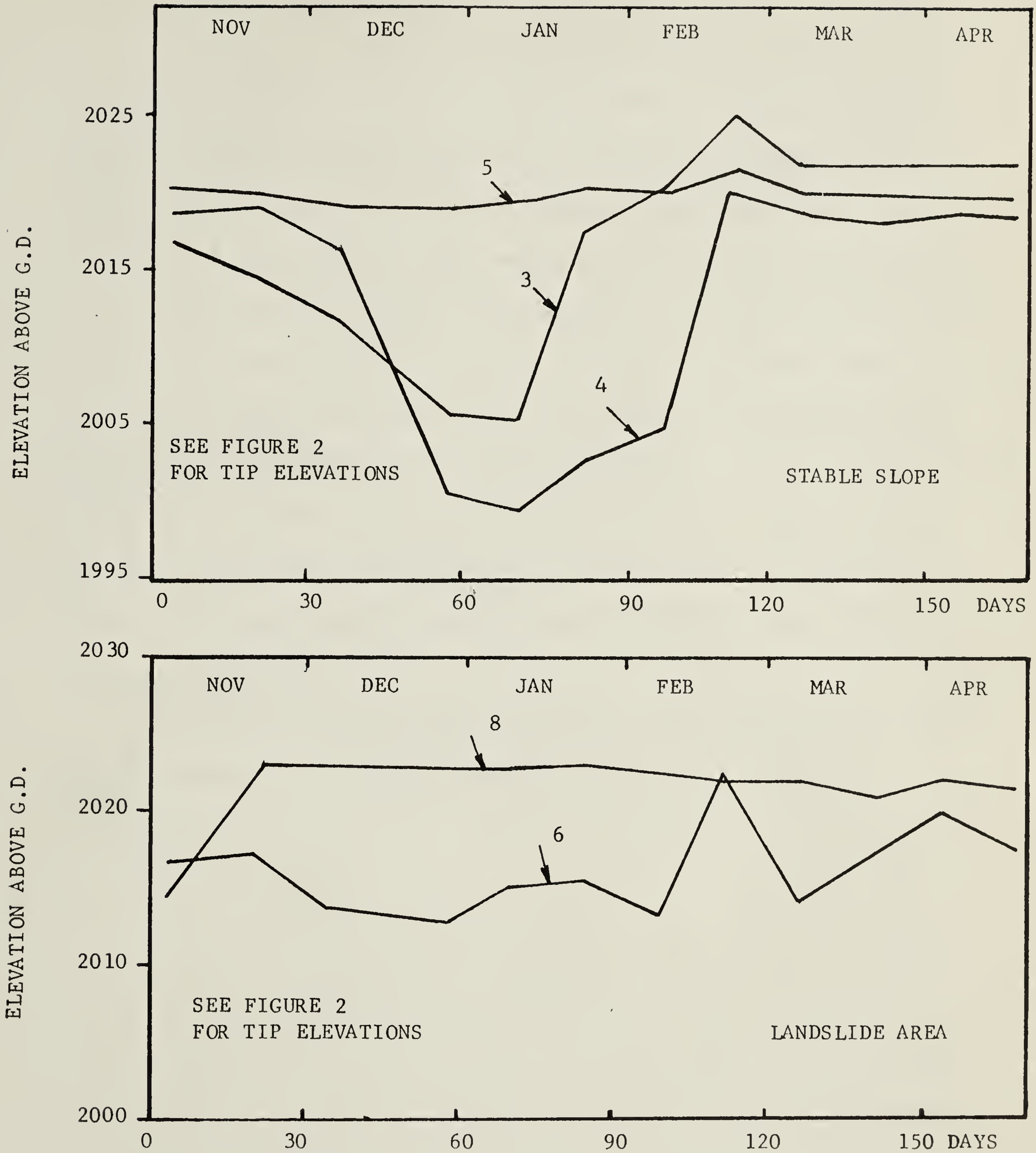


FIGURE 7 PIEZOMETER READINGS

3.4 (d) Probewell Installations

Probewells, consisting of 3" aluminum pipe, were installed in seven boreholes at the locations given in FIGURE 2. Where the pipe was required in lengths greater than 30 ft., it was butt-welded as it was lowered into the boreholes. A few inches of coarse gravel was rammed in the bottom of each probewell hole before placing the pipe and the annular space between the pipe and the borehole was filled with wet sand to within a foot of the ground surface. The top foot was filled with cement grout and a cap of cement was formed around the probewell to prevent surface water infiltration. Slope measurements were made at 5 ft. intervals in the probewells in October 1964 and April 1965. An Eastman Single Shot oil-well survey instrument was used for this purpose and is illustrated in PLATE 5. The slope profiles are included in FIGURES 4 and 5, and are discussed in CHAPTER V.

The aluminium pipes were perforated with small holes and served also as standpipes in which groundwater levels were recorded.

CHAPTER IV

LABORATORY TESTING PROGRAM

4.1 General

The laboratory testing program included classification tests, moisture content determinations, mineralogical, physico-chemical and groundwater analyses, shear strength and swelling pressure studies. The results of the testing program are reported in this chapter and discussed in CHAPTER V.

4.2 Classification of Profile Soils

Classification tests on representative samples of the main profile soils gave the results shown in TABLE IV.1. These tests were performed in accordance with the procedures detailed in A.S.T.M. (1958). Additionally, moisture contents were determined for the profile soils at 5 ft. intervals of depth for all the boreholes. Moisture content profiles are included in FIGURES 3, 4 and 5.

4.3 Mineralogical Composition of Profile Soils

The mineralogical composition of the clay fraction of the cohesive soils in the profile were determined using X-ray diffraction methods. With one exception, the results given in TABLE IV.2 were obtained by the Geology Division of the Research Council of Alberta.

4.4 Physico-Chemical Properties

Analyses were performed on some of the profile soils to determine

TABLE IV.1

SUMMARY OF CLASSIFICATION TESTS

Soil	W_L	W_p	W_s	I_p	% Sand	% Silt	% Clay	A*
Glacio-lacustrine	NP	NP	NP	NP	66	25	9	--
Glacial Till	37.5	17.5	15.1	20.0	33	37	30	0.67
Dark Brown Clay Shale	45.2	21.9	16.4	23.3	21	34	45	0.52
Grey Clay Shale	41.8	20.5	17.8	21.3	24	46	30	0.71
Bentonitic Clay Shale	216.3	32.7	15.2	183.6	17	31	52	3.55
Bentonitic Sand-stone	91.5	27.5	22.5	69.0	50	27	23	3.00
Bentonite	213.9	59.5	17.0	154.4	5	3	92	1.68

*Activity (Skempton, 1953)
NP Non-plastic

TABLE IV.2

MINERALOGICAL COMPOSITION OF

CLAY FRACTION OF PROFILE SOILS

Soil	% Montmorillonite	% Illite	% Kaolinite and/or Chlorite
Glacial Till	50	30	10
Dark Brown Clay Shale	80	10	5
Grey Clay Shale	60	20	10
Bentonitic Sandstone	50	30	5
Bentonite*	100	--	--
Bentonitic Clay Shale	100	--	--

These results are considered accurate within $\pm$ 10%

* This result was obtained by Prof. R.E. Grim, Research Professor of Geology, University of Illinois, Urbana, at the request of Dr. E.W. Brooker

their cation exchange capacity and exchangeable cations. These analyses were carried out by the Soil Science Department of the University of Alberta using flame photometer methods and gave the results shown in TABLE IV.3.

TABLE IV.3

PHYSICO-CHEMICAL PROPERTIES

OF PROFILE SOILS

Soil	Cation Exchange Capacity me/100gm	Ca^{++} me/100gm	Mg^{++} me/100gm	K^{+} me/100gm	Na^{+} me/100gm	Range of o/o Na^{+} of Total*
Glacial Till	13.9	26.5	5.1	0.5	0.2	0.6 1.4
Dark Brown Clay Shale	39.1	26.5	4.6	1.3	18.7	36.5 48.0
Grey Clay Shale	32.6	28.3	9.3	1.5	3.8	8.9 11.6
Bentonitic Sandstone	16.4	24.4	-	0.4	8.8	26.0 54.0
Bentonite	56.2	29.8	1.3	0.9	34.5	52.0 61.5
Bentonitic Clay Shale	52.8	28.4	3.9	1.5	35.5	51.0 67.3

*Minimum and maximum values given

4.5 Groundwater Analyses

Chemical analyses were performed on samples of groundwater obtained from most of the dry-drilled boreholes. These analyses were performed by Prof. P.H. Bouthillier, Professor of Civil Engineering, University of Alberta, and are presented in TABLE IV.4.

TABLE IV.4

GROUNDWATER ANALYSES

Borehole No.	pH	Suspended Solids ppm	Alkalinity -equivalent HC_{03} ppm	Ca^{++} + Mg^{++} ppm	Na^{+} ppm
3	7.6	420	410	140	0
4	7.7	1050	1020	36	304
5	7.4	1400	725	252	179
6	7.6	1400	480	224	203
7	8.1	980	780	22	297
D1	7.3	840	660	109	152
D2	8.7	910	900	144	136
G1	7.7	1330	715	426	0

4.6 Shear Strength Studies

4.6 (a) Unconfined Compression Tests

Unconfined compression tests were performed on 3" "undisturbed"

Shelby samples obtained from boreholes G1 and D3. These borehole profiles were considered representative of the soils in the slide area. The results of these are shown in FIGURES 4 and 5, and TABLE V.1. A series of unconfined compression tests on 1cm diameter and 1" long specimens from a block sample of bentonite gave an average value for q_c of 0.90 kg/cm^2.

4.6 (b) Triaxial Shear Strength Tests

Four series of consolidated undrained triaxial tests with pore pressure measurements were performed using Geonor equipment (Andresen et al., 1957) on the important profile soils to obtain peak shear strength parameters in terms of effective stresses. The test specimens used were 1.4" diameter "undisturbed" core samples of dark brown clay shale and bentonite from borehole 3, and "undisturbed" and remoulded samples of bentonitic clay shale from the slide area. The samples were fitted with filter paper side-drains. Before the samples were sheared they were subjected to a back pressure to minimise volume changes in the pore pressure measuring system. The Mohr envelopes obtained from the tests are presented in FIGURES 8 and 9 and the results are summarised in TABLES IV.5 to IV.8 inclusive. The results are discussed in CHAPTER V.

4.7 Swelling Pressure Test

A constant volume swelling pressure test was performed on an "undisturbed" sample of bentonitic clay shale from the slide area, using distilled water as the immersing fluid. The results of the test are shown graphically in FIGURE 10, and discussed briefly in CHAPTER V. This test was performed by Mr. J. Goris, Graduate Student, University of Alberta, using a one-dimensional consolidation apparatus.

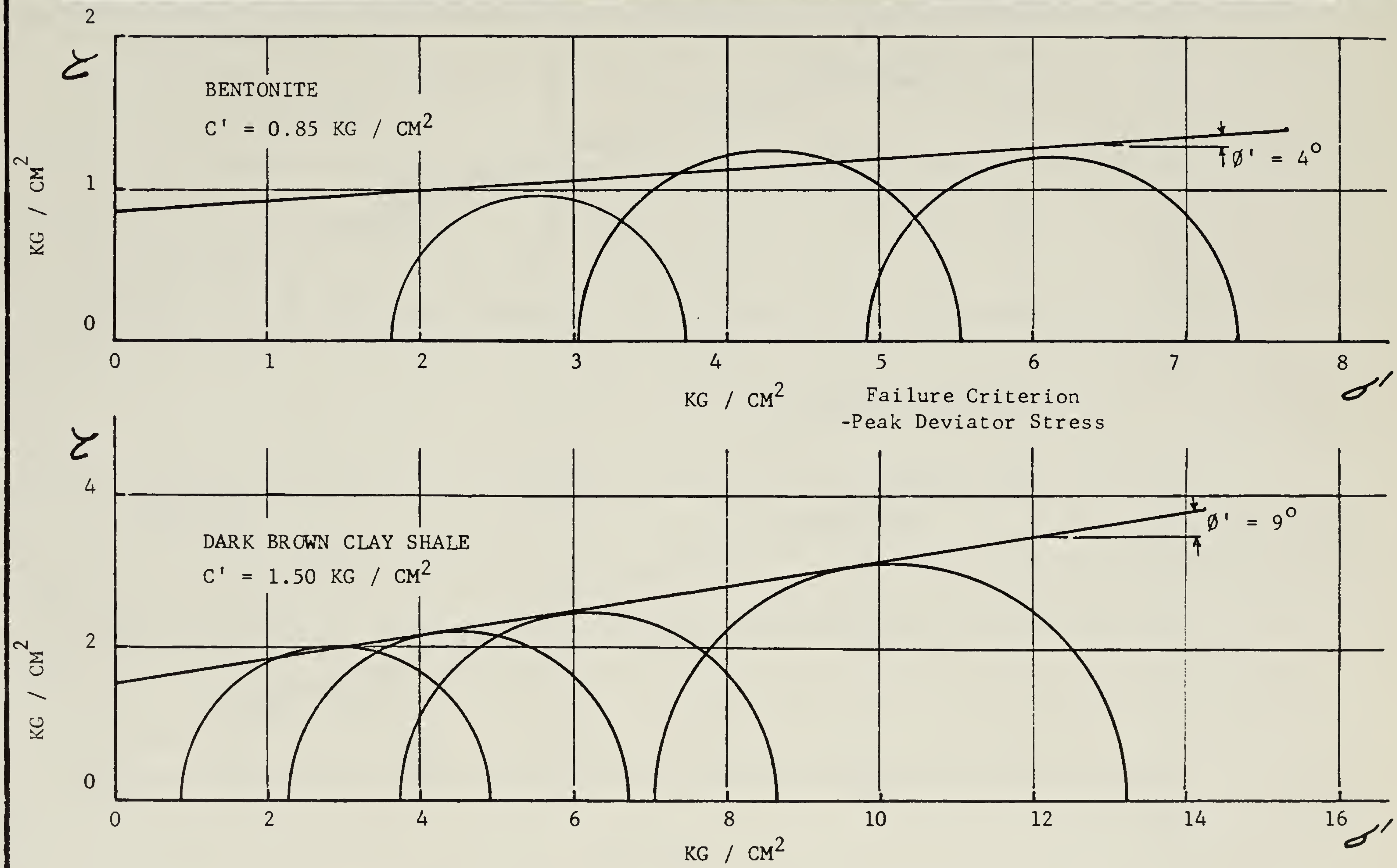
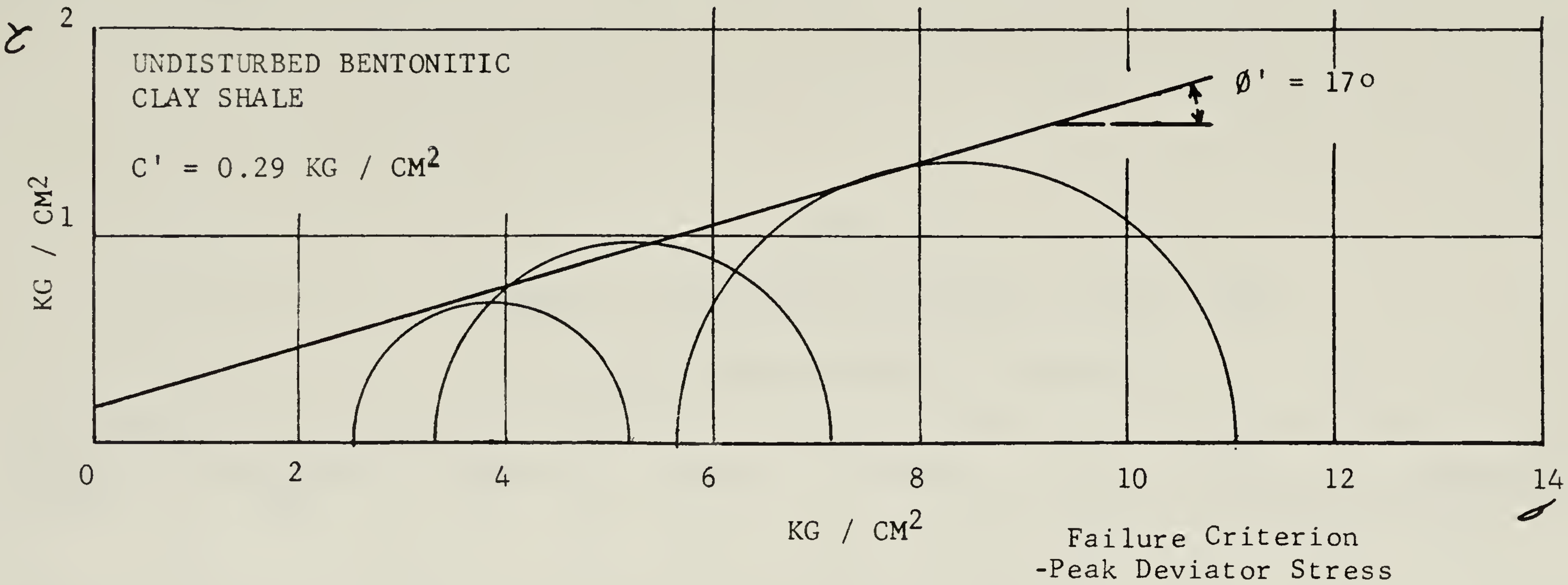


FIGURE 8 MOHR DIAGRAMS FOR EDMONTON FORMATION IN TERMS OF EFFECTIVE STRESS



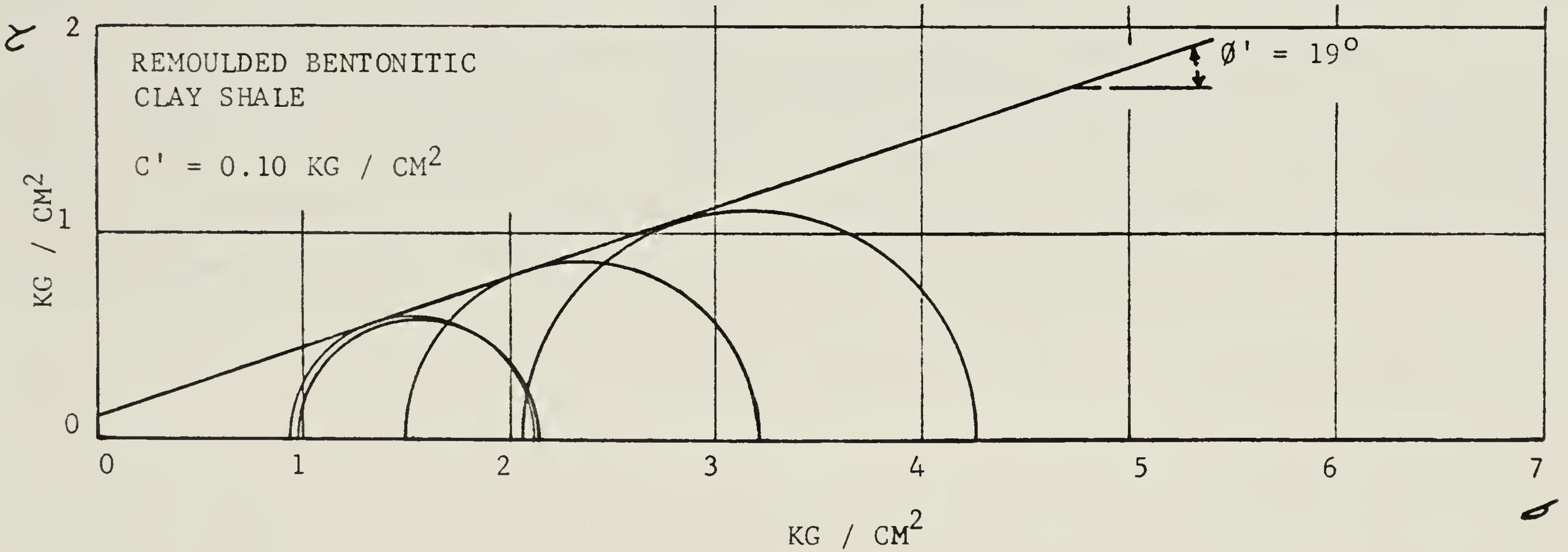


FIGURE 9 MOHR DIAGRAMS FOR EDMONTON FORMATION IN TERMS OF EFFECTIVE STRESS

TABLE IV.5

SUMMARY OF CONSOLIDATED UNDRAINED TRIAXIAL TESTS

ON UNDISTURBED SAMPLES OF BENTONITE

Sample No.	Borehole No.	Depth ft.	Initial % S_r	Final % S_r	σ_{1f} kg/cm^2	σ_{3f} kg/cm^2	Strain at Failure %	u_f kg/cm^2	B
1*	3	94.50	96.5	96.1	7.32	4.89	4.0	1.11	.54
2	3	93.25	110.8	101.0	5.52	3.00	5.53	0.20	.06
3	3	93.75	101.4	100.8	3.71	1.81	9.23	0.19	.06

* Data Sheets included in APPENDIX A.

TABLE IV.6

SUMMARY OF CONSOLIDATED UNDRAINED TRIAXIAL TESTS

ON UNDISTURBED SAMPLES OF DARK BROWN CLAY SHALE

Sample No	Borehole No.	Depth ft.	Initial S_r %	Final S_r %	σ_{1f} kg/cm^2	σ_{3f} kg/cm^2	Strain at Failure %	u_f kg/cm^2	B
4	3	95.0	92.40	95.8	8.61	3.71	8.5	3.71	0.41
5	3	92.5	109.90	94.30	6.70	2.25	2.25	-0.09	0.02
6*	3	87.0	93.25	96.80	13.23	7.02	6.03	0.98	--
7	3	87.5	94.80	93.43	4.88	0.88	5.0	0.12	0.19

*Data Sheets included in APPENDIX A.

TABLE IV.7

SUMMARY OF CONSOLIDATED UNDRAINED TRIAXIAL TESTS

ON UNDISTURBED SAMPLES OF BENTONITIC CLAY SHALE

Sample No.	Borehole No.	Depth ft.	Initial S_r %	Final S_r %	σ_{1f} kg/cm^2	σ'_{3f} kg/cm^2	Strain at Failure %	u_f kg/cm^2	B
8	4	20	95.9	92.3	11.03	5.63	7.0	1.47	0.03
9	D3	21	97.3	97.0	5.19	2.52	2.50	-0.38	0.04
10*	6	2.5	87.3	93.7	7.18	3.29	5.75	1.86	0.10

*Data sheets included in Appendix "A"

TABLE IV.8

SUMMARY OF CONSOLIDATED UNDRAINED TRIAXIAL TESTS

ON REMOULDED SAMPLES OF BENTONITIC CLAY SHALE

Sample No.	Borehole No.	Depth ft.	Initial S_r %	Final S_r %	σ_{1f} kg/cm^2	σ_{3f} kg/cm^2	Strain at Failure %	u_f kg/cm^2	B
11	G1	7 1/2 to 17 1/2	49.4	97.6	2.14	0.98	2.75	0.52	0.92
12	G1		53.6	97.0	3.21	1.50	3.00	1.00	0.91
13*	G1		50.67	96.7	4.26	2.07	3.50	1.43	0.72
14#	4	10	96.3	93.5	2.13	0.94	12.0	0.16	0.84

* Data sheets included in Appendix "A"
\# Naturally remoulded sample from stable slope

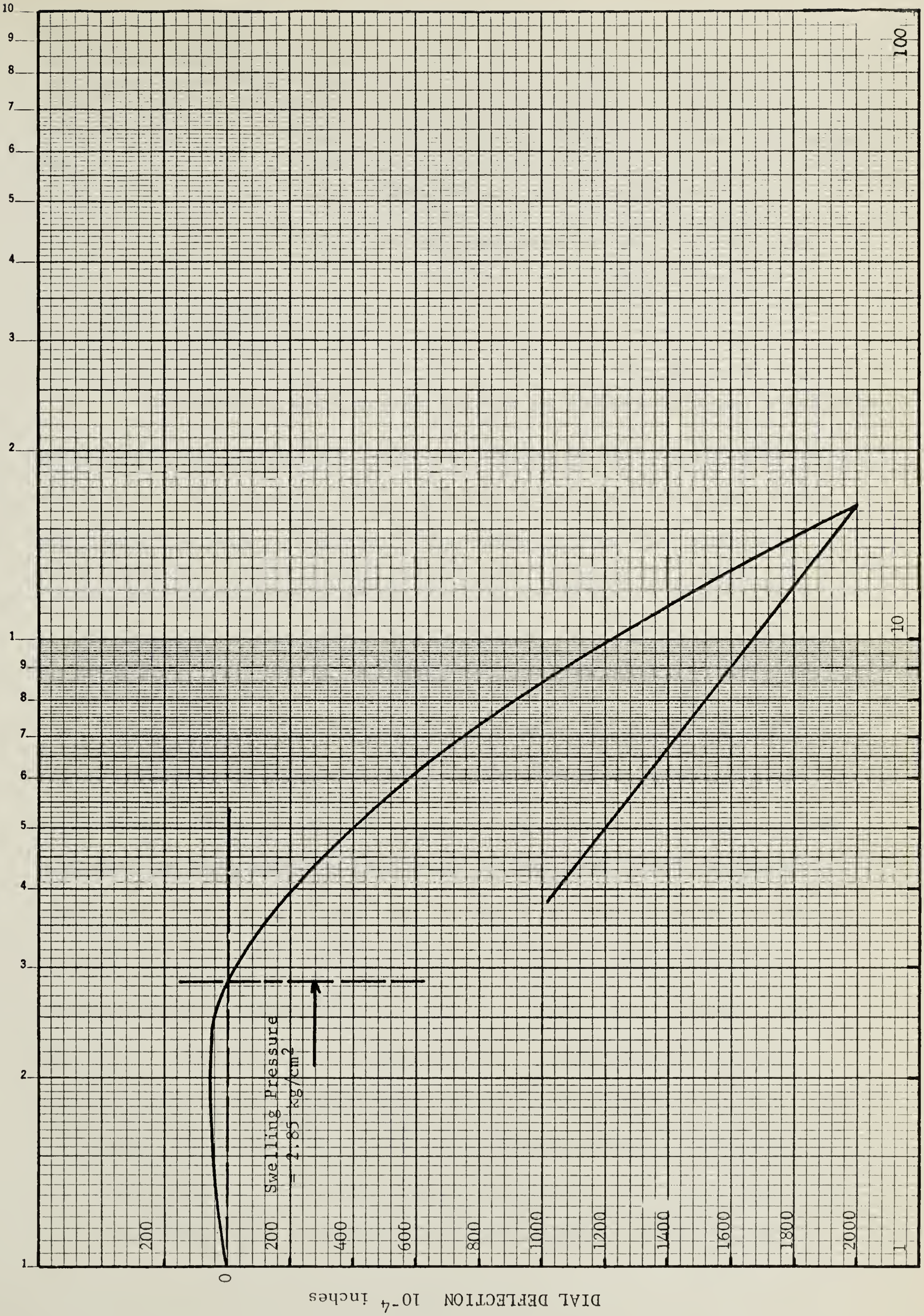


FIGURE 10 SWELLING PRESSURE TEST ON UNDISTURBED BENTONITIC SHALE

CHAPTER V

DISCUSSION OF FIELD AND LABORATORY DATA

5.1 Site Investigation

5.1 (a) Soil Profile

The stratigraphic profiles given in FIGURES 3, 4 and 5 indicate non-uniform soil conditions. This may be due in part to the mode of deposition of the various strata and also to the previous landslides which have occurred in the area. The Edmonton Formation comprises several soil types some of which are only a few inches in thickness. In the sections of the formation revealed by the site investigation, there are several fissured shales underlain by three types of bentonitic soil, namely, bentonite, bentonitic clay shale and bentonitic sandstone. These soils are interspersed with very thin layers of coal. It is evident from the profiles that the bentonitic soils have a large influence on the stability conditions of the river bank. In the unweathered state, as recovered from borehole 3, the bentonite is dark green in colour and contains hair-sized fissures. When exposed to water it becomes yellow in colour and has a soapy consistency. The bentonitic clay shale is a varved deposit consisting mainly of pure bentonite with thinly layered silt, carbonaceous material and fine sand. When wetted this clay shale closely resembles the wetted pure bentonite and it is difficult to differentiate one from the other. In the natural state, the bentonitic sandstone is a hard competent rock, but when

[illegible]

wetted, it rapidly reduces to a sandy bentonitic slurry. The laminated nature of the Edmonton Formation and unique characteristics of the soils within it, cause difficulties with soil identification if orthodox soft drilling methods are employed for site investigation. The only efficient means of obtaining a soil profile accurately under such conditions is by continuous coring. This technique is expensive, particularly where site access is difficult.

5.1 (b) Piezometric Study

Time did not permit a comprehensive study of the groundwater conditions or the plotting of an accurate flow net. However, sufficient piezometric data were obtained which were satisfactory for the purposes of stability analyses when combined with the information concerning groundwater levels determined during drilling.

The elevations and locations of the piezometer tips are shown in FIGURES 2 and 3, and the recordings are given in FIGURE 7. Piezometers 3, 4 and 5 were installed in the stable slope adjacent to the landslide since it was considered that the groundwater flow pattern in this slope would closely approximate the flow pattern in the landslide area before failure of the river bank. Piezometers 6 and 8 were installed in the landslide area to enable a comparison to be made of the piezometric conditions in the two zones. Piezometers 3 and 5 were installed in coal layers and 4, 6 and 8 in clay shales.

Reference to FIGURE 7 shows that piezometers 3, 4 and 6 indicated a dissipation of pore pressure for the first 73 days from November 1st., 1964 to January 12th., 1965. During this period, it may be expected that

pore pressures were returning to pre-installation equilibrium. The piezometers in the stable slope then showed a build up in pore pressure for the next 40 days, reaching a maximum on February 21st., 1965. Reference to APPENDIX "A", which gives climatalogical data for Edmonton during the winter months of 1965, indicates that this rise in pore pressure coincided with the second and longest period during the winter when the temperatures were below zero degrees F. Pore water pressures were erratic during this period within the landslide area as would be expected due to the disruption of the original groundwater flow pattern. Pore pressures throughout the river bank began to dissipate at the beginning of April; however, they began to build up again near the end of April and are slowly increasing at the present time. (May 1965). It is anticipated that the pore pressures will dissipate during the coming summer months under the normal climatic conditions of the area. It will be noted from FIGURE 3 that piezometers 3 and 4 are located one above the other in borehole 4. Tip 3 is located 6 ft. below tip 4 in a layer of coal, while tip 4 is in a layer of dark brown clay shale. Reference to FIGURE 7 shows that the recorded pore pressures in piezometer 3 ranged from 2 ft. to 4 1/2 ft. of water greater than those of piezometer 4 during the months of March and April, 1965, indicating higher pore pressures in the shale than in the coal. These differences in elevation head indicate that the distribution of pore water pressure in the river bank may not be hydrostatic but it is considered more likely that they may be due to the differences in permeability of the soils concerned. It is possible that the piezometer in the shale has a longer response time than the piezometer in the coal and it is hoped that this factor will be checked by continuing pore

pressure observations in the two piezometers in future months.

In May 1965, groundwater levels were determined in the probewells within the landslide area. In all cases, the water elevations were well below those indicated by piezometers in the stable slope. For example, in probewell G4, the elevation of the water was 2013.4 ft. above Geodetic Datum whereas, at the same time, the recorded pore pressures in boreholes 5 and 8 were 2019.9 and 2021.8 ft. above Geodetic Datum, respectively. Since the volume of each probewell is relatively high and its intake factor is very low, it is to be expected that lower pore pressures would be indicated by the probewells than those measured with the hydraulic piezometers.

The piezometers appeared to function satisfactorily throughout one of the coldest winters on record. Since four of the piezometers are below the level of the gauge house, extra care is required when de-airing the tips; however, all the tips are less than 20 ft. below the gauge house and are located in boreholes where the maximum vertical distance from the piezometric surface to the ground surface is less than 29 ft.

5.1 (c) Probewell Study

The Eastman inclinometer gave a qualitative indication of the probewell profiles. Some movement of the landslide took place between the months of October 1964 to April 1965, and probewells D1, D2 and G3 were blocked off by this movements thus giving a positive indication of at least the upper limit of the slip surface. The positions of blockage in these probewells have been plotted in FIGURE 14 and have been used for the purposes of stability analysis of the landslide.

The absolute magnitudes and directions of movement of the landslide cannot be ascertained as the tops of the probewells have moved since the time of their installation and these were not accurately located by triangulation at the time of their installation.

5.1 (d) Apparent Failure Mechanism of the River Bank

The piezometric study showed that pore water pressures in the stable slope increased by as much as 20 ft. of water during the winter of 1964-65. This is considered to have been caused by the freezing of seepage layers at the surface of the river bank since icings form on and extend into these layers when temperatures are below zero degrees F and flow through the bank is prevented or severely restricted. See PLATE 9.

From June to August 1964, the level of the North Saskatchewan River rose at least 10 ft. at the LeSueur site and much soil was removed from the toe of the landslide due to the increased flow.

On the basis of these observations, it is postulated that the LeSueur Landslide was initiated by an annual process of toe erosion during the summer months followed by pore pressure build up during the winter months. For the purposes of stability analysis, major failure will be considered as having occurred in January 1963 when tension cracks first appeared in the top of the river bank and the beginnings of movement were noted. It is possible that such a mechanism of failure is a prime reason for many of the landslides which occur along the banks of the North Saskatchewan River in the Edmonton area.

5.2 Laboratory Testing Program

5.2 (a) Classification Tests and Physico-Chemical Properties

The Atterberg and hydrometer tests give an indication of the

PLATE 9. ICING ON LOWER EAST FLANK APRIL 1965.

physical properties of the profile soils. A comparison of the various properties of the profile soils given in TABLES IV.1, IV.2 and IV.3 reveals some interesting relationships which, for ease of interpretation are reproduced in TABLE V.1.

TABLE V.1

RELATIONSHIPS BETWEEN PLASTICITY INDEX AND

PHYSICO-CHEMICAL PROPERTIES OF PROFILE SOILS

Soil	Montmorillonite % of clay sizes*	I_p	Range of $\%Na^{++}$ of Total Exchange Cations
Glacial Till	50	20.0	0.6 - 1.4
Grey Clay Shale	60	21.3	8.9 - 11.6
Dark brown clay shale	80	23.3	36.5 - 48.0
Bentonitic Sandstone	50	69.0	26.0 - 54.0
Bentonite	100	154.0	52.0 - 61.5
Bentonitic clay shale	100	183.6	51.0 - 67.3

*Less than .002 mm

It can be seen from TABLE V.1 that in general, the plasticity index of the soils increases with both the percentage of montmorillonite and the range of percent sodium cations of the total exchangeable cations.

Although the significance of these relationships is not immediately apparent, these findings may be of use at some future date when further similar research work is performed.

The glacial till, dark brown and grey clay shales are defined

as inactive and the bentonitic soils as active (Skempton, 1953).

5.2 (b) Groundwater Analyses

The results of the groundwater analyses may indicate that, during many years of leaching, calcium cations have replaced sodium cations in some of the profile soils. Reference to TABLE IV.4 shows that in bore-holes 3 and G1, the groundwater contained appreciable amounts of calcium and magnesium but no sodium. At these locations, it is considered that the main groundwater flow before the LeSueur Landslide was probably within the Saskatchewan sands and gravels. In all the other boreholes, the groundwater contains various amounts of calcium and magnesium and in all cases some sodium. It is quite evident that the groundwater flow in the lower sections of the river bank was through many strata including the upper sections of the Edmonton Formation before the landslide occurred.

Reference to FIGURE 3, borehole 3, shows that the dark brown clay shale and the grey clay shale generally overlie the bentonitic soils in the bedrock formation. These shales probably have a higher permeability and may have been leached to a greater extent than the bentonite and bentonitic clay shale. In the natural state, the bentonitic sandstone is predominantly a calcium soil and it is considered unlikely that this stratum has been leached at all during recent geologic time. Since there is a distinct lieklihood that the soils within the Edmonton Formation could have been either calcium or sodium saturated before seepage through them commenced, the aforementioned observations must be considered speculative. The studies of Thomson (1963) indicate that the replacement of sodium cations by calcium would result in a strength increase of the soils concerned.

5.2 (c) Shear Strength Studies

5.2 (c) (i) Unconfined Compression Tests

The results of the unconfined compression tests for boreholes G1 and D3 have been included in FIGURES 4 and 5, however, for ease of interpretation these are reproduced in TABLE V.2.

The average unconfined compressive strengths for the soils in the Edmonton Formation, based on the results given in TABLE V.2 and previous test results are as follows:

Bentonite (W = 80%)	0.90 kg/cm^2
Dark Brown Clay Shale (W = 36%)	2.80
Carbonaceous Shale (W = 45%)	3.50
Bentonitic Clay Shale (W = 29%)	4.30
Grey Clay Shale (W = 34%)	5.50
Bentonitic Sandstone (W = 30%)	8.40

The range of strength for the shales in the formation appears to be relatively small and the strength of the bentonite is appreciably lower at a moisture content of 80% than any of the other materials. However, the bentonite recovered from borehole 3 had a natural moisture content of 46% and it is to be expected that the unconfined compressive strength at this moisture content would have been higher than that quoted. The coal layers in the formation are very thin and of variable quality; therefore, no value for the unconfined compressive strength (q_u) of the coal is given. The value of q_u given for the bentonitic sandstone is for the material in the natural state.

TABLE V.2

UNCONFINED COMPRESSION TESTS ON SAMPLES

FROM BOREHOLES G1 and D3

Borehole	Soil-Type	Depth ft.	q_u kg/cm^2	W%
G1	Glacio-Lacustrine	2.5	5.1	11
	Glacio-Lacustrine	7.5	1.4	21
	Till	15.0	2.9	23
	BSS*	17.5	3.8	20
	GS	22.5	5.5	34
	BS	35.0	4.6	39
	BS	37.5	2.7	19
	BST	45.0	9.0	31
	BS	50.0	4.8	31
	BS CS	55.0	5.4	42-25
D3	Slumped CS	2.5	1.3	29
	Coal	5.0	8.3	16
	CS	7.5	3.2	45
	CS	10.0	3.7	34
	BS	12.5	4.6	30
	DBS	17.5	2.8	36
	BST	25.0	7.7	28

*See FIGURE 3 for Legend

5.2 (c) (ii) Consolidated Undrained Triaxial Tests

Unfortunately, time did not permit a comprehensive study of the shear strength characteristics of the Edmonton Formation; however, it is considered that sufficient data were obtained to permit satisfactory stability analyses of the LeSueur Landslide. Peak effective shear strength parameters were determined for the three principal soils encountered in the formation using consolidated undrained triaxial tests with pore pressure measurements. The results of these tests have been summarised in TABLES IV.5 to IV.8 and typical data sheets have been included in APPENDIX "A". It will be noted that some anomalies exist in the summary tables. Generally, the pore pressure reactions (B)[1] for the undisturbed samples are extremely low in spite of the seemingly high initial and final degrees of saturation (S_r). Furthermore, the data sheets show that small negative pore pressures were recorded after the consolidation stages of the tests. The negative pore pressures and low pore pressure reactions probably resulted due to partial saturation and the calculated degrees of saturation may be in error. It is probable that the S_r values are all on the high side for the undisturbed samples. The 1.75" diameter cores from which these samples were prepared were obtained by mud-flush drilling. Consequently, the cores had a thin coating of wetter soil. Unfortunately, there was a limited amount of each soil type available for strength tests due to the small thickness of each stratum and complete sections of core could not be used for determining initial moisture contents. These were determined from core trimmings which obviously had more moisture than the 1.4" diameter test specimens. The final S_r values are thought to be closer to the true values but also on the high side. Henkel and Sowa (1963)

1. The pore pressure parameter B was determined by increasing the cell pressure by 1 kg/cm^2 in one increment, after a back pressure of 1 kg/cm^2 had been applied to the pore water pressure measuring system.

have reported that when the cell pressure is lowered to atmospheric pressure at the end of the consolidated undrained triaxial test, negative pore pressures set up in the test specimen may cause cavitation in the pore pressure measuring system and water may be drawn into the sample through the porous stone. This effect may be accentuated due to the mineralogy of these soils.

It is evident that the remoulded samples were almost fully saturated at the end of the consolidation stages of the tests since the pore pressure reactions were high and no significant negative pore pressures were recorded. The initial degrees of saturation given in the summary table for these samples are considered accurate since an adequate amount of the remoulded soil was available for moisture content determinations.

FIGURES 8 and 9 show the Mohr envelopes in terms of effective stress for the dark brown clay shale, the bentonite and the bentonitic clay shale. Peak deviator stress has been used for the failure criterion and the range of cell pressure has been selected in accordance with the overburden pressure to which each soil has recently been subjected. It will be noted from FIGURE 9 that the shear strength parameters for the remoulded samples of bentonitic clay shale are slightly higher than those for the undisturbed samples. Similar findings were reported by Henkel and Skempton (1955) for the overconsolidated fissured clay at Jacksfield in Shropshire, England. Reference to FIGURE 8 shows that the shear strength parameters for the dark brown clay shale and the bentonite are, respectively, $c' = 1.50$ kg/cm^2 and $\phi' = 9^\circ$; and $c' = 0.85$ kg/cm^2 and $\phi' = 4^\circ$. These values of cohesion are high and the angles of shearing resistance are relatively low particularly for the bentonite. The author cannot offer a

definitive explanation of these unorthodox parameters at this time; however it would appear to be significant that similar shear strength characteristics have been previously reported for a western Canadian soil. Crawford (1964) performed tests on Winnipeg clay using both consolidated undrained and drained triaxial tests. He obtained parameters of $c' = 0.6$ kg/cm^2 and $\emptyset' = 9^{\circ}$ for this soil and it is of interest to note that the stress versus strain and pore pressure versus strain plots which he reported, closely resembled the shapes of those given in APPENDIX "A" for samples 1, 6 and 10. A factor which may be thought significant to the shear strength parameters of these soils is their low permeability. Although the permeabilities of these soils have not been determined in the present study, it is thought that they are all less than 10^{-8} cms/sec. Several investigators have expressed the need for extremely slow rates of testing for such soils to achieve sufficient pore pressure equilisation during the shearing stage of the consolidated undrained triaxial test. e.g. Bishop and Henkel, (1962), Bishop and Blight, (1963). At the present time, it is not known if the effects of strain rates are of practical importance to the shear strength parameters and the subject appears to require further research. It has been assumed for the purposes of stability analysis in the present study that the strain rate is not of practical importance.

Stress path plots for the dark brown clay shale, the bentonite and bentonitic clay shale are given in FIGURES 11 and 12 and moisture content versus log deviator stress relationships for these soils are included in FIGURE 13. FIGURE 11 shows that the samples of dark brown clay shale 7, 5 and 4 behaved as overconsolidated samples during shear

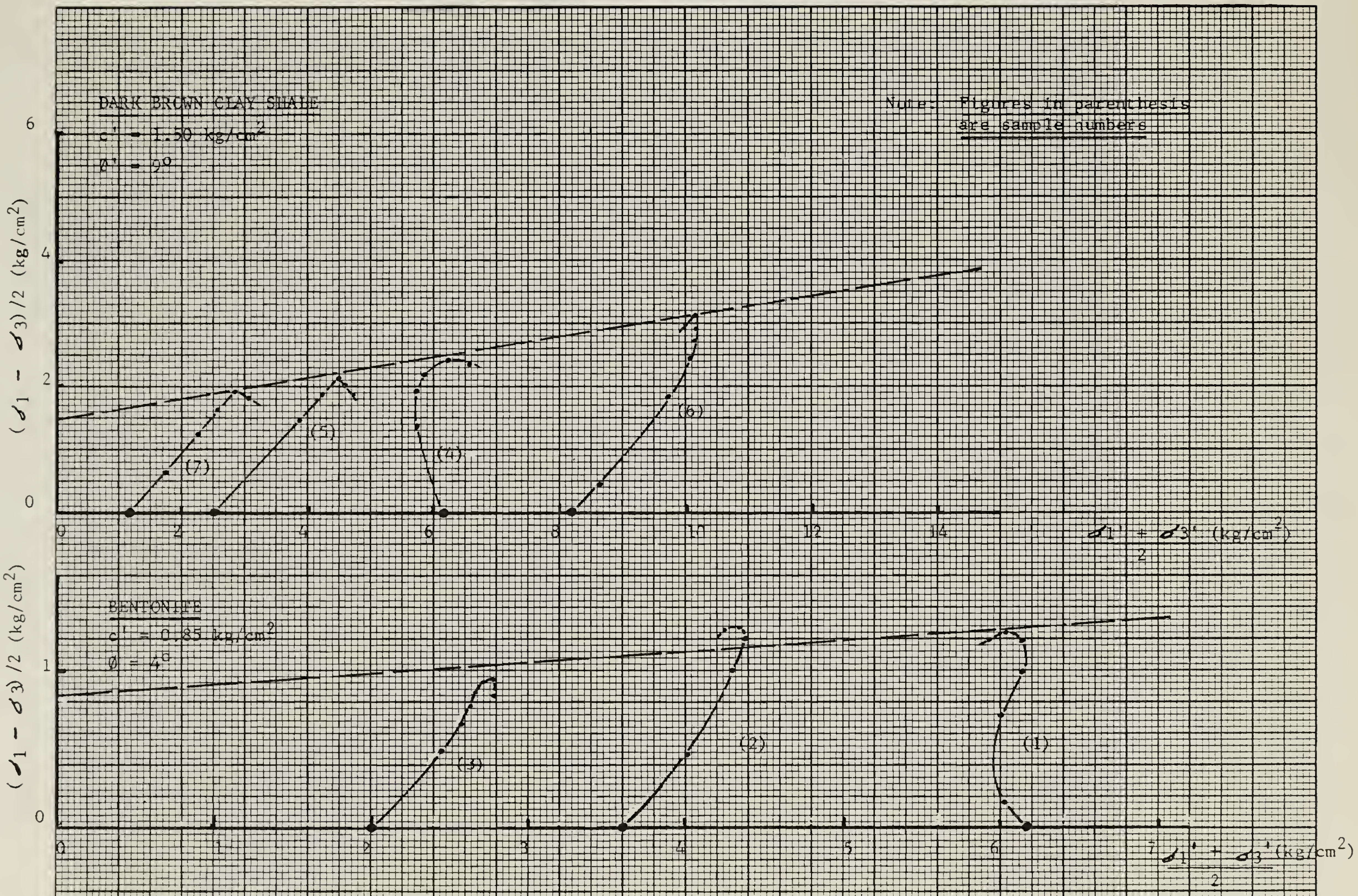


FIGURE 11. STRESS PATH PLOTS FOR CONSOLIDATED-UNDRAINED TRIAXIAL TESTS ON EDMONTON FORMATION SOILS IN TERMS OF EFFECTIVE STRESS

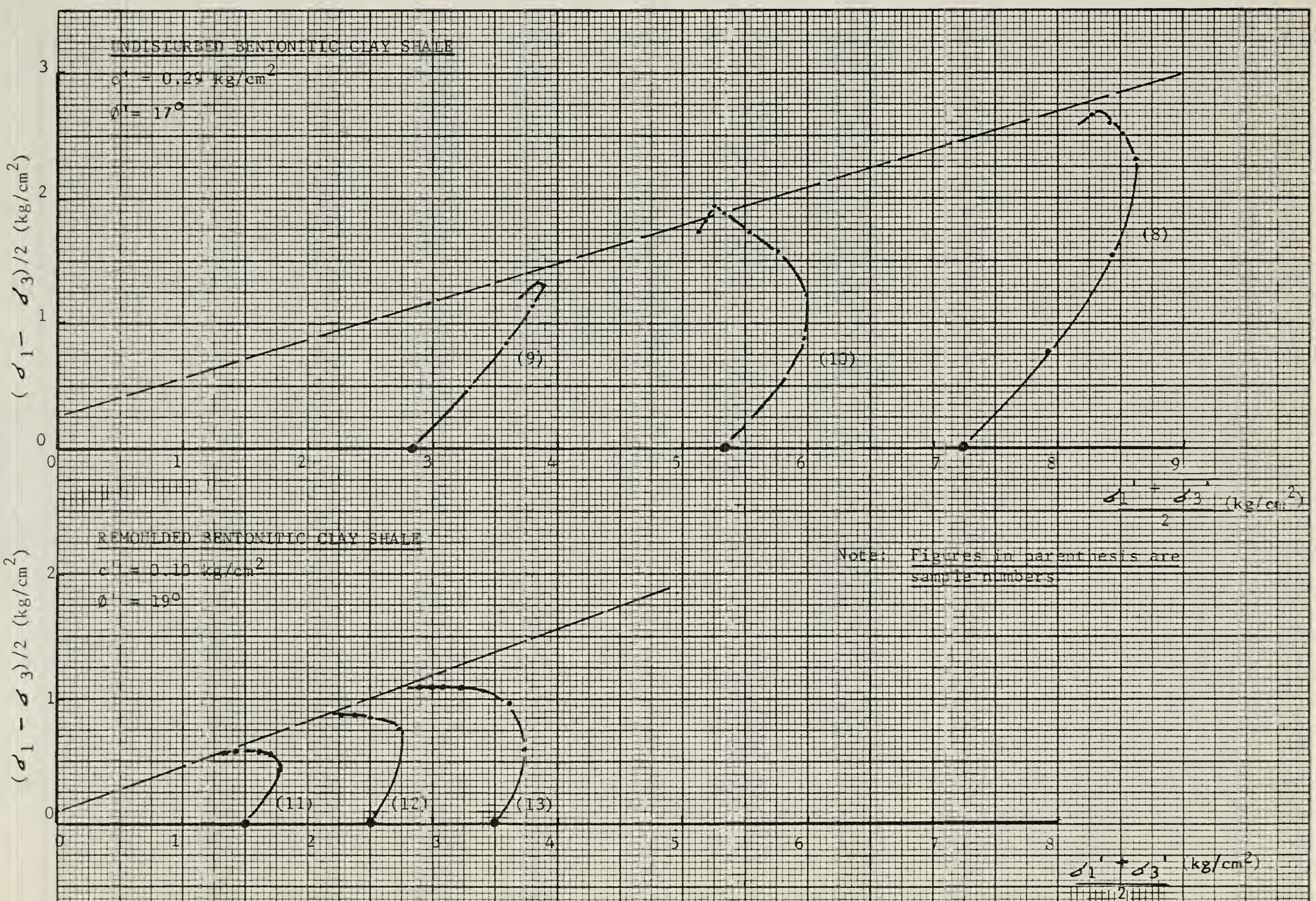


FIGURE 12 STRESS PATH PLOTS FOR CONSOLIDATED-UNDRAINED TRIAXIAL TESTS ON EDMONTON FORMATION SOILS IN TERMS OF EFFECTIVE STRESS

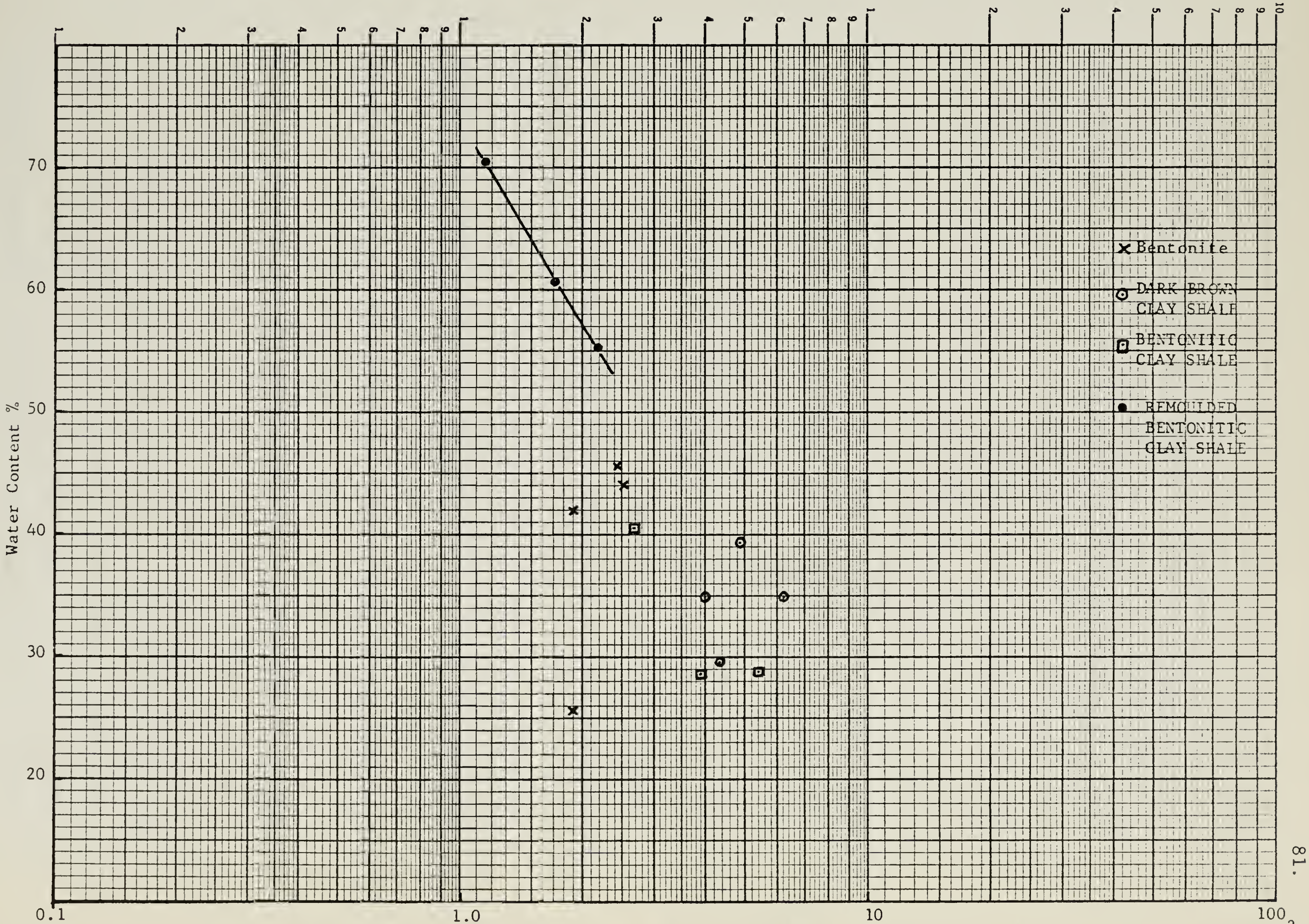


FIGURE 13 MOISTURE CONTENTS vs LOG $(\sigma_1 - \sigma_3)_f$ RELATIONSHIPS FOR EDMONTON FORMATION SOILS

and that sample 6 behaved as a normally consolidated sample. This would appear to indicate that the preconsolidation load for this soil lies between 6 and 8 kg/cm^2. Since this set of samples was obtained from a depth of about 90 ft. where the soil was subjected to an overburden pressure of about 4 1/2 kg/cm^2, the indicated overconsolidation ratio would appear to be less than 2.0. In a similar way, it can be seen that the preconsolidation load for the bentonite appears to lie between 2 and 3.6 kg/cm^2. However, these samples were recently subjected to an overburden pressure of about 5 kg/cm^2 and thus it can only be assumed that the bentonite is particularly sensitive to sample disturbance and that the samples swelled appreciably on removal from the ground. The undisturbed samples of bentonitic clay shale were obtained from a depth of about 20 ft. and FIGURE 12 shows that these samples all behaved as though normally consolidated during the shearing stages of the triaxial tests.

It is evident that there are natural inconsistencies in the three soils tested since there were variations in their dry densities of as much as 6 pounds per cubic foot within only a few inches of depth. Therefore, it is not surprising that, in FIGURE 13, linear relationships have not been established between the moisture content and log deviator stress at failure, for the undisturbed samples. Based on the information given in FIGURES 8, 9, 11, 12 and 13 it is considered that the soils in the Edmonton Formation may have been reworked, since glaciation, by river action or previous landslides and furthermore that their shear strength characteristics are influenced by their clay mineralogy and exchangeable cations. It is also apparent that these soils are sensitive to sample disturbance. At this time, it is not possible to establish any

definitive relationships between the shear strength and physico-chemical properties of the soils; however, it is hoped that the information obtained from the present study will be useful at some future date when further similar research work has been performed.

5.2 (d) Swelling Pressure Test

The swelling pressure of the undisturbed bentonitic clay shale was found to be 2.85 kg/cm^2 using distilled water as the immersing fluid.

CHAPTER VI

STABILITY ANALYSES

6.1 General

Stability analyses have been performed, using several methods to ascertain factors of safety with respect to shear strength for an approximated average slope through the river bank before its failure, and an adjacent stable slope. The analyses of the landslide have necessitated approximations concerning the geometry of the river bank and the pore water pressures in the bank before failure occurred. The geometry of the adjacent stable slope has been accurately determined and piezometer measurements of pore water pressures have been made within it; therefore, it is anticipated that the factors of safety for this slope must lie within the limits of experimental error. A comparison of the factors of safety obtained for the stable slope with those determined for the landslide thus gives a guide to the reliability of the landslide analyses. The stability analyses for the landslide are summarised in tabular form in FIGURE 14 and the analyses of the stable slope in FIGURE 15.

6.2 Selection of Parameters for Analysis

6.2 (a) Shear Strength Parameters

It will be seen from TABLE V.2 that the range of unconfined compressive strength for the various shales in the Edmonton Formation is relatively small. Furthermore, the stratigraphic profiles indicate that

the principal strata encountered in the formation were the dark brown clay shale, the bentonite and the bentonitic clay shale. Shear strength parameters were obtained in terms of effective stress for these soils. See FIGURES 8 and 9. For the purposes of stability analyses, these three soils were considered to give the range of shear strength of the Edmonton Formation and their peak values of cohesion and angles of shearing resistance were arithmetically averaged. Where slip surfaces were assumed tangential to a bentonitic layer, the appropriate parameters for this layer were used with these average values. Since no shear strength tests were performed on samples of the glacial soils it was necessary to assume effective stress shear strength parameters for these. In this regard the author used the work of Soderman, Kenney and Loh (1960) as a guide.

6.2 (b) Pore Water Pressures

It has been assumed that the groundwater flow pattern in the stable slope closely approximated the flow pattern in the landslide area before failure of the river bank. Therefore, all the stability analyses have been based on a phreatic line which has been obtained for the stable slope. Groundwater levels in boreholes 1 and 2, determined during drilling, were used to plot the phreatic line at the top of the river bank and the maximum piezometer measurements in boreholes 4 and 5 were used for this line near the bottom of the bank. Between boreholes 2 and 4, the phreatic line was assumed to be parabolic in shape as shown in FIGURES 14 and 15. It is realised that the assumption that the distribution of pore pressure within the river bank is hydrostatic may be a simplification of the actual conditions but, even taking into account the differences in the piezo-

meter measurements in borehole 4, this assumption does not appear to be grossly in error. A linear distribution of uplift pressure was assumed for the simplified sliding block analyses.

6.3 Stability Analyses of the Landslide

6.3 (a) General

It has been considered that major failure of the landslide occurred in January 1963 when tension cracks appeared in the top of the river bank and the beginnings of movement of the bank were noted. Furthermore, it has been considered that pore pressures in the landslide area, at this time, had reached magnitudes close to those recorded in the stable slope in February 1965.

The positions of the landslide slip surface determined in probe-wells D1, D2 and G3 have been plotted in FIGURE 14 and indicate, with the shape of the crown scarp, that the shape of the slip surface approximated that of a log spiral having its base at a mean elevation of 2002 ft. above Geodetic Datum. The geometry of the landslide resembled that of a sliding block which appeared to have its base within a layer of bentonitic soil. Since samples of this bentonitic soil were not recovered by continuous coring methods in the landslide area, it is not known whether the soil was bentonitic clay shale or bentonite. Consequently, analyses of the landslide have been performed assuming the shear strength parameters of each soil type for comparative purposes.

6.3 (b) Preliminary Stability Analyses

Five slip circles numbered 1 to 5 (FIGURE 14) were considered for preliminary stability analyses according to Bishop (1954), using

average shear strength parameters for the Edmonton Formation[1] and assumed parameters for the glacial soils[2]. The lowest factor of safety was obtained from these analyses using Circle 4 and since this circle closely approximated the actual slip surface it was used for subsequent analyses.

6.3 (c) Analyses for Average Conditions

FIGURE 16 shows the variation of the factor of safety with the average effective cohesion of the Edmonton Formation assuming that the average angle of shearing resistance is 10^{o}. This relationship has been established by performing analyses according to Bishop (1954) for Circle 4, and indicates that for the LeSueur Landslide, the average shear strength parameters for the Edmonton Formation would be $\bar{c}' = 0.61$ kg/cm^2 and $\bar{\phi}' = 10^{o}$ to give a factor of safety of unity[3].

6.3 (d) Analyses Approximating Actual Conditions

Four methods of effective stress analysis have been used to assess the factor of safety of the river bank for the time of its failure, using approximately similar geometry in each case and appropriate shear strength parameters[4]. The analysis which most closely approximates the geometry of

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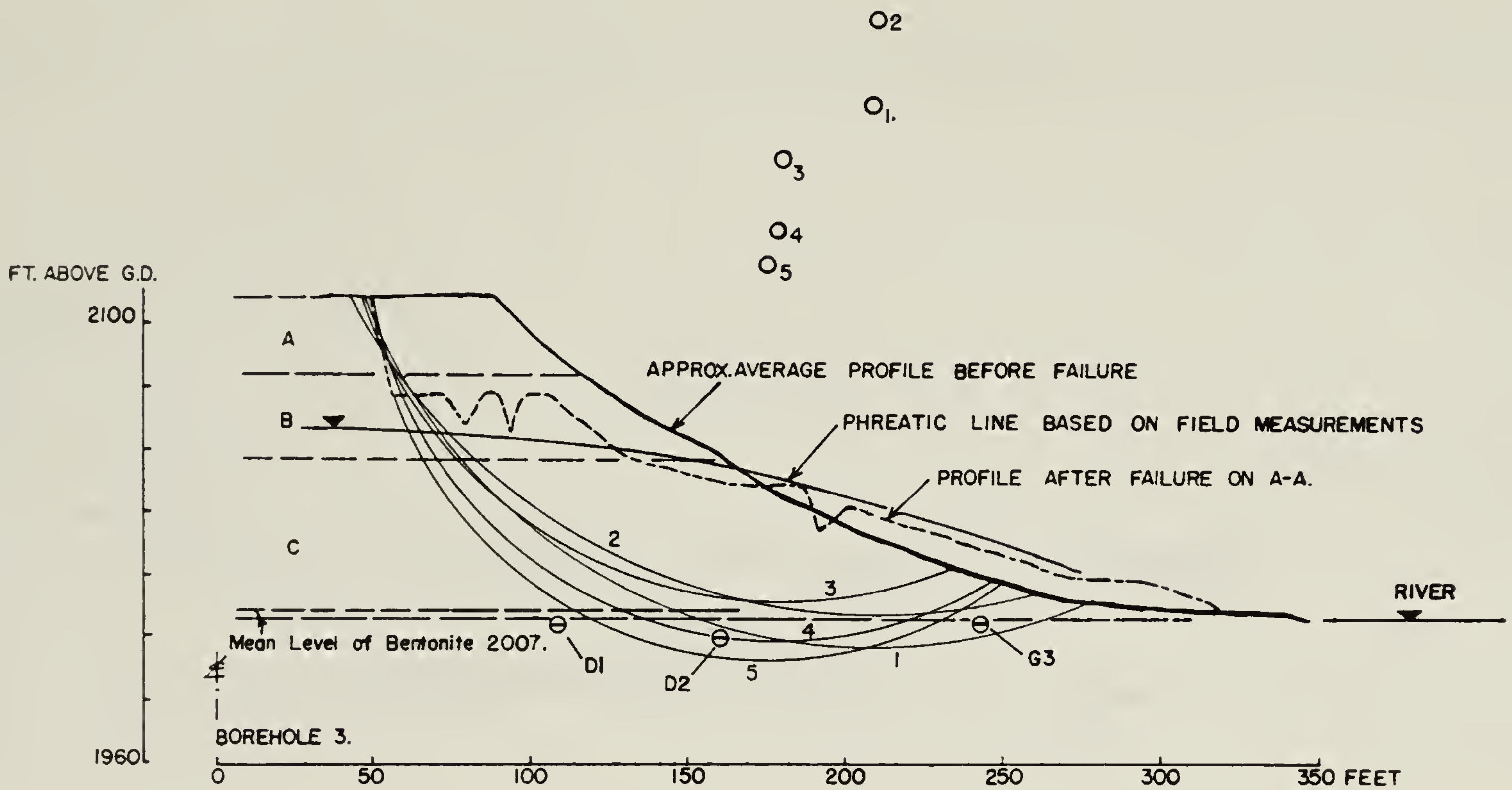
1. $\bar{c}' = 0.85$ kg/cm^2 (1741 psf) and $\bar{\phi}' = 10^{o}$ for DBS, BS and B or as shown in FIGURE 14.

2. $\bar{c}' = 0$ and $\phi' = 22^{o}$, glacio-lacustrine deposits, $c' = 0.15$ kg/cm^2 $\phi' = 26^{o}$ till.

3. These parameters may be suitable for stability computations in the Edmonton area where a comprehensive site investigation is not feasible.

4. $\bar{c}' = 0.85$ kg/cm^2, $\bar{\phi}' = 10^{o}$ BS, DBS and B, $\bar{c}' = 0.1$ kg/cm^2, $\phi' = 19^{o}$ BS, or as shown in FIGURE 14.

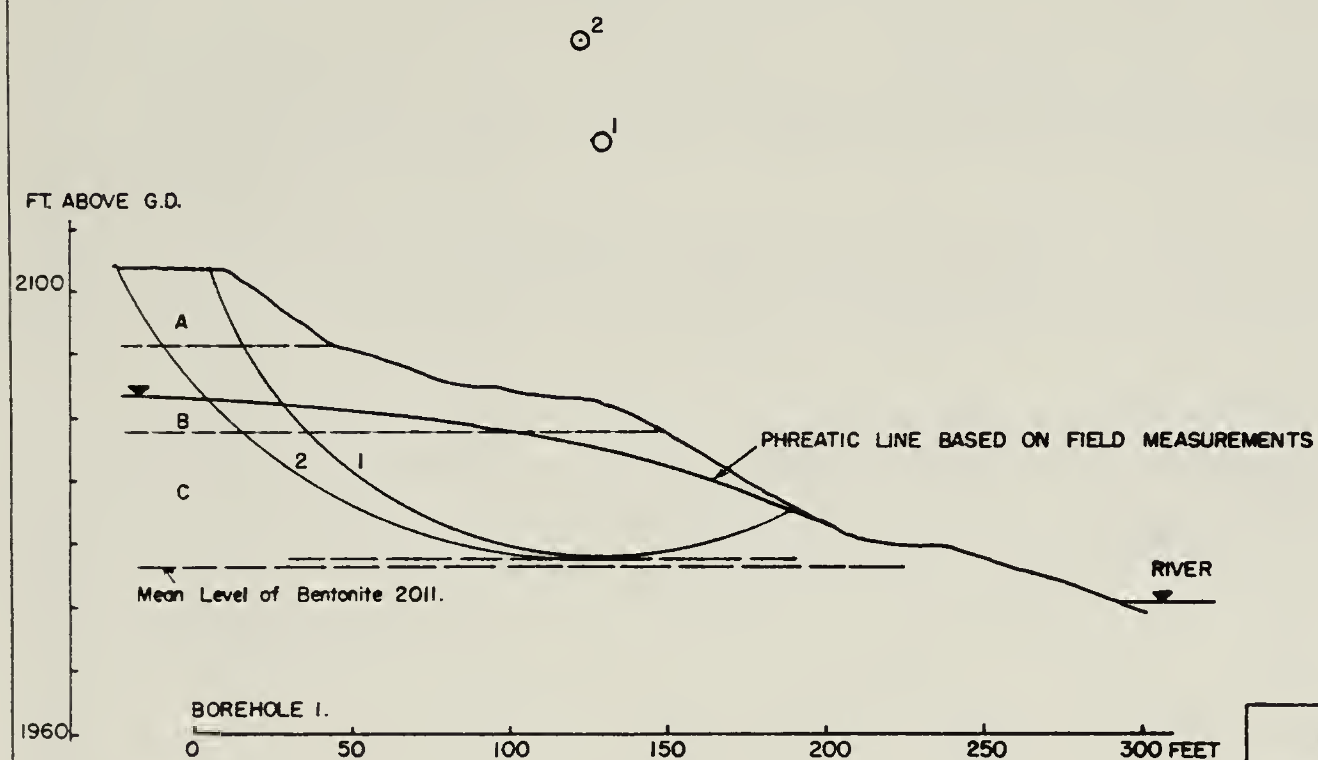


LEGEND	
A	GLACIAL LACUSTRINE DEPOSITS
B	TILL
C	EDMONTON FORMATION
θ	POSITIONS OF SLIP SURFACE

METHOD	CIRCLE	NUMBER OF SLICES	F	STRENGTH PARAMETERS
BISHOP (1954).	1	10	1·28	A. C'= 0 ϕ'= 22° B. C'=307psf ϕ'=26° C. C' =1536psf ϕ'= 10°
HENKEL & SKEMPTON (1955).	1	10	0·35	A. and B as above C. C' = 0 ϕ' = 10°
BISHOP (1954).	2	10	1·39	A. and B as above C. C' = 1741 ϕ' = 13° = 1741 ϕ' = 4°(Bentonite).
BISHOP (1954).	3	10	1·29	A. and B as above C. C' = 1536psf ϕ'=10° = 1741 ϕ' = 4°(Bentonite).
BISHOP (1954).	4	11	1·14	A and B as above C. C'=1536psf ϕ' = 10°
HENKEL & SKEMPTON (1955).	4	11	0·38	A and B as above C. C' = 0 ϕ' = 10°
BISHOP (1954).	5	10	1·18	As Circle 4
JANBU (1954).	4	—	4·70	Average C = 3·7 Kg/cm^2 – C = 0·9 Kg/cm2 (Bentonite).

METHOD	BANK HEIGHT	AVERAGE SLOPE	F	STRENGTH PARAMETERS
SIMPLIFIED SLIDING BLOCK	105'	2:1	0·97 0·65	C. C' = 1741psf ϕ' = 4° (B) C. C' = 410psf ϕ' = 18° (BS)
BISHOP-MORGENSTERN (1960).	98'	2:1	1·40	Weighted Averages C' = 1458psf ϕ' = 13°
BISHOP-MORGENSTERN (1960).	105'	2:1	1·30	As above.
BISHOP (1954)	CIRCLE 4	SLICES 11	0·92	A & B As shown opposite C. C' = 1741psf ϕ' = 10° C' = 205psf ϕ' = 19°
NOTE:- All analyses assume given phreatic line which is based on field observations and pore pressure measurements for stable slope. Bishop's method (1954) – $(X_n - X_{n+1})$ terms ignored. Bishop & Morgenstern method (1960). – Some inaccuracy $C'/\gamma H > 0{\cdot}10$. Simplified Sliding Block method – Sliding surface assumed on Bentonite layer with linearly distributed uplift pressure.				
HENKEL & SKEMPTON (1955)	4	11	0·45	A & B As shown C. C' = 0 ϕ' = 10° C' = 0 ϕ' = 19°

FIGURE 14. STABILITY ANALYSES OF LANDSLIDE.



LEGEND	
A	GLACIAL LACUSTRINE DEPOSITS
B	TILL
C	EDMONTON FORMATION

NOTE: All analyses assume given phreatic line which is based on field observations and pore pressure measurements.

Bishop's method (1954) — (Xn-Xn+1) terms ignored.

Bishop & Morgenstern method (1960) — Some inaccuracy $C'/\gamma H > 0{\cdot}01$.

Simplified Sliding Block method — Sliding surface assumed on Bentonite layer with linearly distributed uplift pressure.

METHOD	BANK HEIGHT	AVERAGE SLOPE	F	STRENGTH PARAMETERS
BISHOP-MORGENSTERN (1960)	95' 106'	2·5 : 1	1·56 1·50	Weighted Averages $C' = 1332$psf $\phi' = 14°$ $C' = 1458$psf $\phi' = 13°$
SIMPLIFIED SLIDING BLOCK	10	2·5 : 1	1·00	A & B As below C. $C' = 1741$psf $\phi' = 10°$ Slip sfce $C' = 410$psf $\phi' = 18°$
SIMPLIFIED SLIDING BLOCK	95'	2·5 : 1	1·37	A. $C' = 0$ $\phi' = 22°$ B. $C' = 307$psf $\phi' = 26°$ C. $C' = 1741$psf $\phi' = 4°$ sfce.
	CIRCLE	No. of SLICES		
BISHOP (1954).	1	12	1·97	A. and B. as above. C. $C' = 1720$psf $\phi' = 12{\cdot}5°$ $= 1536$ $\phi' = 4{\cdot}5°$
BISHOP (1954).	2	14	1·73	As Circle 1.
HENKEL & SKEMPTON (1955)	2	14	0·60	A and B as above. C. $C' = 0$ $\phi' = 12{\cdot}5°$ $= 4{\cdot}5°$
JANBU (1954)	1	—	5·80	C = 3·7 (AVERAGE). TOTAL STRESSES.

FIGURE 15. STABILITY ANALYSES OF STABLE SLOPE.

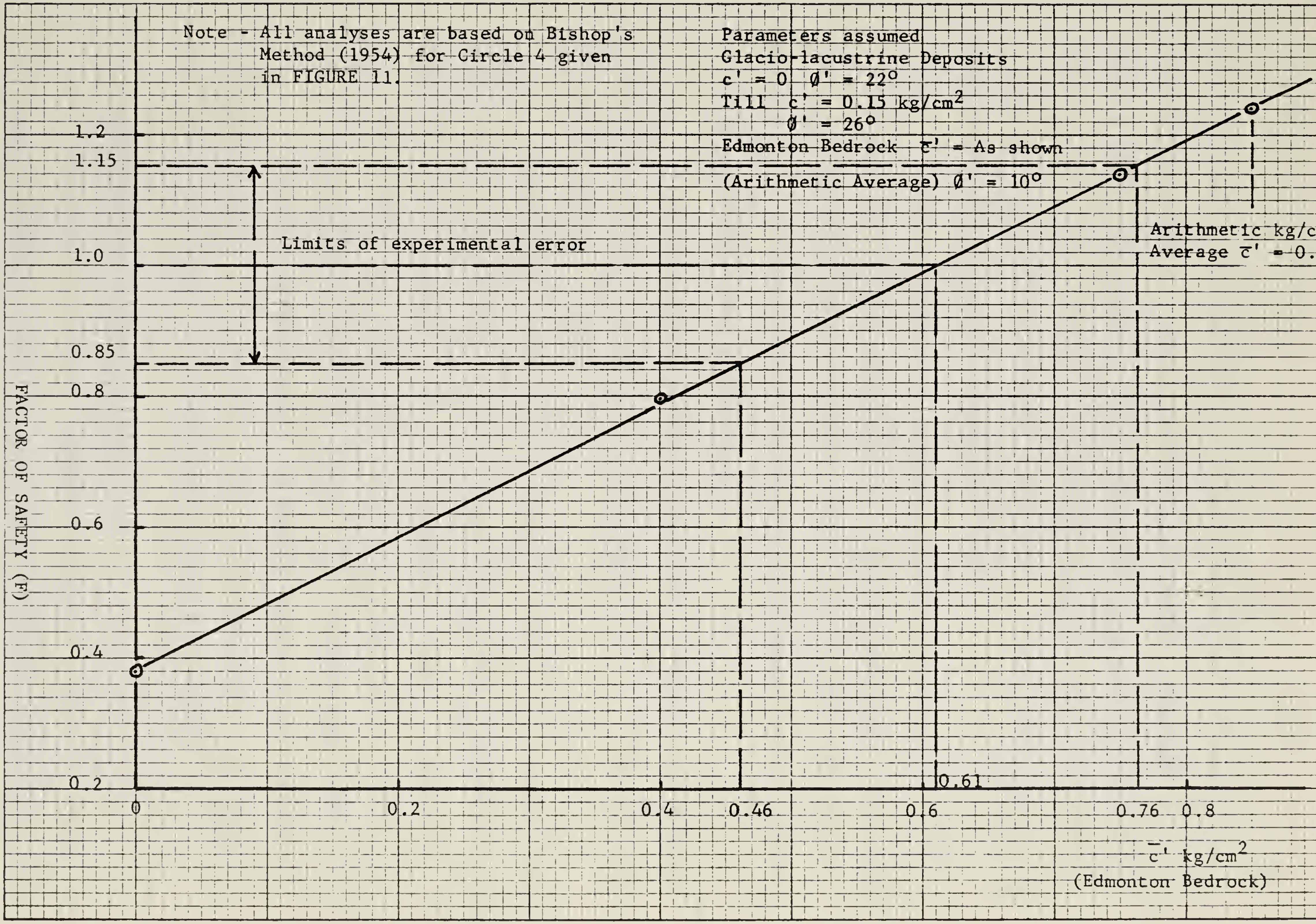


FIGURE 16 LANDSLIDE STABILITY ANALYSES

VARIATION OF "F" WITH AVERAGE EFFECTIVE COHESION OF EDMONTON BEDROCK

the landslide is the simplified sliding block analysis. Assuming an average slope of 2 : 1 and a bank height of 105 ft., this method of analysis gives a factor of safety of 0.97 assuming that the base of the sliding block is in a layer of bentonite and 0.65 assuming that the base is in a layer of bentonitic clay shale. These factors of safety appear to be within tolerable limits of experimental error.

Bishop's method (1954) using Circle 4 gives factors of safety of 0.92 assuming the circle tangential to a layer of bentonitic clay shale and 1.13 assuming it tangential to a layer of bentonite. This method appears to give a satisfactory assessment of the stability conditions of the river bank before its failure.

A "zero-cohesion" analysis according to Henkel and Skempton (1955) gives a factor of safety using circle 4 of 0.45. This result is obviously grossly in error.

Bishop's and Morgernstern's coefficients for an average slope of 2 : 1 and a bank height of 105 ft. using weighted average shear strength parameters for the complete soil profile give a factor of safety of 1.30. This result is in error due to the incorrect assumption of linear variation of the factor of safety (F) with $c'/\gamma H$ values greater than 0.10.

Unfortunately, time did not permit a more precise method of stability analysis to be used for sliding block geometry. It is hoped that such an analysis can be performed at some future date.

6.4 Stability Analyses of the Stable Slope

Boreholes 1, 2 and 4 (FIGURE 3) indicate a layer of bentonite at a mean elevation of 2011 ft. above Geodetic Datum through the stable

slope. A sliding block analysis, for an average slope of 2.5:1 and a bank height of 95 ft. with this layer as the base of potential movement, gives a factor of safety of 1.37. Two analyses according to Bishop (1954) for circles 1 and 2 (FIGURE 15) tangential to this layer of bentonite give factors of safety of 1.97 and 1.73. It would thus appear that this layer is not a criterion for future failure; however, if it is assumed that the potential slip surface is at the same elevation as the slip surface for the landslide within a lower layer of bentonitic soil, a simplified sliding block analysis gives a factor of safety of 1.00. This result, although on the low side, is considered realistic for the stability conditions which exist along the North Saskatchewan River Valley in this area. It may also indicate that the factors of safety for the simplified analyses of the landslide are on the low side.

CHAPTER VII

CONCLUSIONS

7.1 General

In the light of the available literature pertinent to the LeSueur Landslide, and in view of the results obtained from this research study, it appears that the following conclusions are justified.

7.2 Conclusions Arising from the Literature

1. If it is accepted that a 15% error in the calculation of the factor of safety is within the limits of experimental error, then an effective stress analysis, using peak shear strength parameters and measured field pore water pressures is satisfactory for the stability analysis of natural slopes in normally consolidated and overconsolidated intact clays.

2. Existing concepts of the insitu behavior of overconsolidated fissured clays are based on a relatively small number of effective stress analyses of natural slopes in these clays. In the author's opinion, in some cases, the site investigations upon which these analyses were based appear to have been inadequate since, the positions of the slip surfaces and pore water pressures were assumed rather than measured.

3. Three modifications to the effective stress shear strength parameters have been used for assessing the stability conditions of natural slopes in overconsolidated fissured clays. These were reported

by Henkel and Skempton (1955), Hardy, Brooker and Curtis (1962) and Skempton (1964). Respectively, these modifications concern the "zero-cohesion" concept, the introduction of the swelling pressure and the use of the residual strength. In view of the preceding conclusion, the worth of these modifications is questionable.

4. In the author's opinion, inadequate piezometric studies of natural slopes in overconsolidated fissured clays may have led to factors of safety on the high side of unity. Hence, these piezometric studies may have been an important contributory factor to the existing beliefs that there are wide discrepancies between laboratory test results and the actual values of the shear strength of these soils.

7.3 Conclusions Arising from the Present Study

1. Although the soils in the Edmonton Formation have been subjected to very large overburden pressures and surcharges during their geologic histories, they appear to currently behave as though slightly overconsolidated. It is possible that these soils may have been reworked since their deposition by river action or previous landslides; however, there is also some evidence from the study to suggest that their shear strength characteristics are influenced by their clay mineralogy and type of exchangeable cations.

2. It is postulated that the LeSueur Landslide was initiated by an annual process of toe erosion by the North Saskatchewan River during the summer months followed by a maximum pore pressure build up during the winter months due to the freezing of seepage layers at the surface of the river bank. It is considered that failure occurred in January 1963,

when tension cracks first appeared in the river bank and the beginnings of movement were noted.

3. The geometry of the landslide resembled a sliding block having its base in a layer of bentonitic soil at an elevation of about 2002 ft. above Geodetic Datum. It is not known if this soil was bentonitic clay shale or bentonite since samples of the bentonitic soil were not recovered by continuous coring methods in the landslide area.

4. Stability analyses, using peak shear strength parameters in terms of effective stress and pore water pressures based on piezometer measurements and recorded groundwater levels gave a satisfactory assessment of the stability conditions of the river bank. The following results were obtained:

(i) A simplified sliding block analysis of the landslide, which most closely approximated the actual conditions of failure, gave a factor of safety of 0.65 assuming the base of the landslide was in a layer of bentonitic clay shale and 0.97 assuming the base in a layer of bentonite.

(ii) Analyses of the landslide according to Bishop (1954) gave a factor of safety of 0.92 assuming a circle closely approximating the shape of the slip surface tangential to a layer of bentonitic clay shale, and 1.13 assuming the same circle tangential to a layer of bentonite.

(iii) A simplified sliding block analysis of the stable slope, assuming a potential sliding surface at the same elevation as that determined for the landslide, gave a factor of safety of 1.00.

The factors of safety obtained from each method of analysis are considered to be within the limits of experimental error and a factor

of safety of unity for the stable slope, although on the low side, is thought to be realistic for the conditions which exist along the banks of the North Saskatchewan River in this area.

CHAPTER VIII

RECOMMENDATIONS

The author presents the following recommendations for future studies concerning the LeSueur Landslide based on the procedures and results of the present study.

1. That recordings of pore water pressure fluctuations in the river bank be continued at the LeSueur site during the coming summer and winter months to confirm the present findings. Particular attention should be given to the recordings for piezometers 3 and 4 to ascertain whether the tips have different response times or if the distribution of pore water pressure in the river bank is consistently non-hydrostatic.

2. That the piezometers installed at the LeSueur site be used later to determine the insitu permeabilities of the soils in which they are installed by means of a falling head permeability test. This can be done by connecting a water reservoir of known elevation to one of the piezometer leads.

3. That a more precise stability analysis in terms of effective stress be performed for the landslide for sliding block geometry. The modified Swedish method of slices or Janbu's extension (1956) of Bishop's method would be suitable.

4. That further physico-chemical analyses and shear strength studies be performed on the undisturbed samples remaining from the present study. Several 4 1/2" diameter cores of the dark brown and

bentonitic clay shales are available and these are suitable for residual shear strength tests. It is anticipated that the results of such tests would corroborate the results obtained using remoulded samples in the consolidated undrained triaxial tests, i.e. that residual shear strength parameters for the LeSueur Landslide soils are close to the peak values. Furthermore, it may be possible to establish some definitive relationships between the shear strength characteristics of the soils, their mineralogy and type of exchangeable cations.

5. That a definitive study be undertaken to examine the effects of strain rate on the shear strength parameters of undisturbed samples of western Canadian soils. Orthodox triaxial test procedures could be employed with a sufficient back pressure during the consolidation stages of the tests to ensure complete saturation of the test specimens.

LIST OF REFERENCES

LIST OF REFERENCES

Andresen, A.L., Bjerrum, L., Dibiago, E., and Kjaernsli, B., "Triaxial Equipment developed at the Norwegian Geotechnical Institute" NGI Publication No. 21, Oslo, 1957.

American Society for Testing Materials "Procedures for Testing Soils", 1958.

Bayrock, L.A. and Hughes, G.M., "Surficial Geology of the Edmonton District Alberta". Research Council of Alberta Province. Report 62-6., 1962.

Bishop, A.W., "The Use of the Slip Circle in the Stability Analysis of Slopes". Proceedings European Conference on Stability of Earth Slopes, Stockholm 1:1-13, 1954. Also published in Geotechnique Volume 5:1:7, 1955.

Bishop, A.W., "Some Factors Controlling the Pore Pressures Set Up During the Construction of Earth Dams". Proceedings of the 4th International Conference on Soil Mechanics and Foundation Engineering, London, August 1957.

Bishop, A.W. and Bjerrum, L., "The Relevance of the Triaxial Test to the Solution of Stability Problems". ASCE Research Conference on Shear Strength of Cohesive Soils, Boulder, Colorado, June 1960.

Bishop, A.W. and Blight, G.E., "Some Aspects of Effective Stress in Saturated and Partly Saturated Soils". Geotechnique Volume 13:3:177, 1963.

Bishop, A.W., and Henkel, D.J., "Pore Pressure Changes During Shear in Two Undisturbed Clays". Proceedings of the Third International Conference on Soil Mechanics and Foundation Engineering, 1:94, 1953.

Bishop, A.W. and Henkel, D.J., "The Measurement of Soil Properties in the Triaxial Test", Edward Arnold Publishers Ltd., London, Second Edition, 1962.

Bishop, A.W., Kennard, M.F., and Penman, A.D.M., "Pore Pressure Observations at Selset Dam", Conference on Pore Pressure and Suction in Soils, London, 1960.

Bishop, A.W. and Morgenstern, N., "Stability Coefficients for Earth Slopes". Geotechnique Volume 10:4:129, 1960.

Bjerrum, L., and Kjaernsli, B., "Analysis of Some Norwegian Natural Clay Slopes". Geotechnique Volume 7:1:1, 1957.

Crawford, C.B., "Some Characteristics of Winnipeg Clay", Canadian Geotechnical Journal, Volume 1, Number 4, 1964.

Gibson, R.E., and Henkel, D.J., "Influences of Duration of Tests at Constant Rate of Strain on Measured "drained" strength". Geotechnique Volume 4:1:6, 1954.

Hardy, R.M., "Engineering Problems Involving Preconsolidated Clay Shales". Transactions of the Engineering Institute of Canada, September, 1957.

Hardy, R.M., Brooker, E.W., and Curtis, W.E., "Landslides in Over-consolidated Clays", The Engineering Journal, June, 1962.

Henkel, D.J., "Investigations of two Long-term Failures in London Clay Slopes at Woodgreen and Northolt", Proceedings of the Fourth International Conference on Soil Mechanics and Foundation Engineering, London, 1957.

Henkel, D.J., and Skempton, A.W., "A Landslide at Jacksfield, Shropshire in a Heavily Overconsolidated Clay". Geotechnique, Volume 5:2:131, 1955.

Henkel, D.J., and Sowa, V.A., "The Influences of Stress History on Stress Paths in Undrained Triaxial Tests on Clay", ASTM Special Technical Publication No. 361, 1963.

Hutchinson, J.N., "A Landslide on a Thin Layer of Quick Clay at Furre, Central Norway". Geotechnique Volume 11:2:69, 1961.

Janbu, N., "Stability Analysis of Slopes with Dimensionless Parameters". Harvard Soil Mechanics Series No. 46, 1954.

Janbu, N., "Stability Calculations for Fills, Cuts and Natural Slopes", NGI, Publication No. 16, 1956, Published in Norwegian with Summary in English.

May, D.R. and Brahtz, J.H.A., "Proposed Methods of Calculating the Stability of Earth Dams", Transactions of the Second Congress on Large Dams 4:539, 1936.

Penman, A.D.M., "A Field Piezometer Apparatus". Geotechnique 6:2:57, 1956.

Peterson, R., Iverson, N.L. and Rivard, P.J., "Studies of Several Dam Failures on Clay Foundations". Proceedings of the Fourth International Conference on Soil Mechanics and Foundation Engineering, London, 1957.

Peterson, R., Jaspar, J.L., Rivard, P.J. and Iverson, N.L., "Limitations of Laboratory Shear Strength in Evaluating Stability of Highly Plastic Clays", ASCE Research Conference on Shear Strength of Cohesive Soils, Boulder, Colorado, 1960.

Ringheim, A.S., "Experience with the Bearpaw Shale at the South Saskatchewan River Dam". Transactions of the Eighth Congress on Large Dams, Edinburgh, 1964.

Sevaldson, R.A., "The Slide in Lodalen, October 6, 1954". Geotechnique Volume 6:4:167, 1956.

Skempton, A.W., "A Slip in the West Bank of the Eau Brink Cut". Journal I.C.E. 24:267-287, 1945.

Skempton, A.W., "The Colloidal Activity of Clays". Proceedings of the Third International Conference on Soil Mechanics and Foundation Engineering. Volume 1, 1953.

Skempton, A.W., "Long Term Stability of Clay Slopes", Geotechnique Volume 14:2:75, 1964.

Skempton, A.W. and Brown J.D., "A Landslide in Boulder Clay at Selset, Yorkshire". Geotechnique Volume 11:4:280, 1961.

Skempton, A.W., and Delory, F.A., "Stability of Natural Slopes in London Clay". Proceedings of the Fourth International Conference on Soil Mechanics and Foundation Engineering, London, 1957.

Soderman, L.G., Kenney, T.C. and Loh, A.K., "Geotechnical Properties of Glacial Clays in Lake St. Clair Region of Ontario". Proceedings of the Fourteenth Canadian Soil Mechanics Conference, October, 1960.

Taylor, D.W., "Fundamentals of Soil Mechanics", J. Wiley and Sons Inc., New York, 1948.

Terzaghi, K., "Stability of Slopes on Natural Clay". Proceedings of First International Conference on Soil Mechanics and Foundation Engineering, 1936.

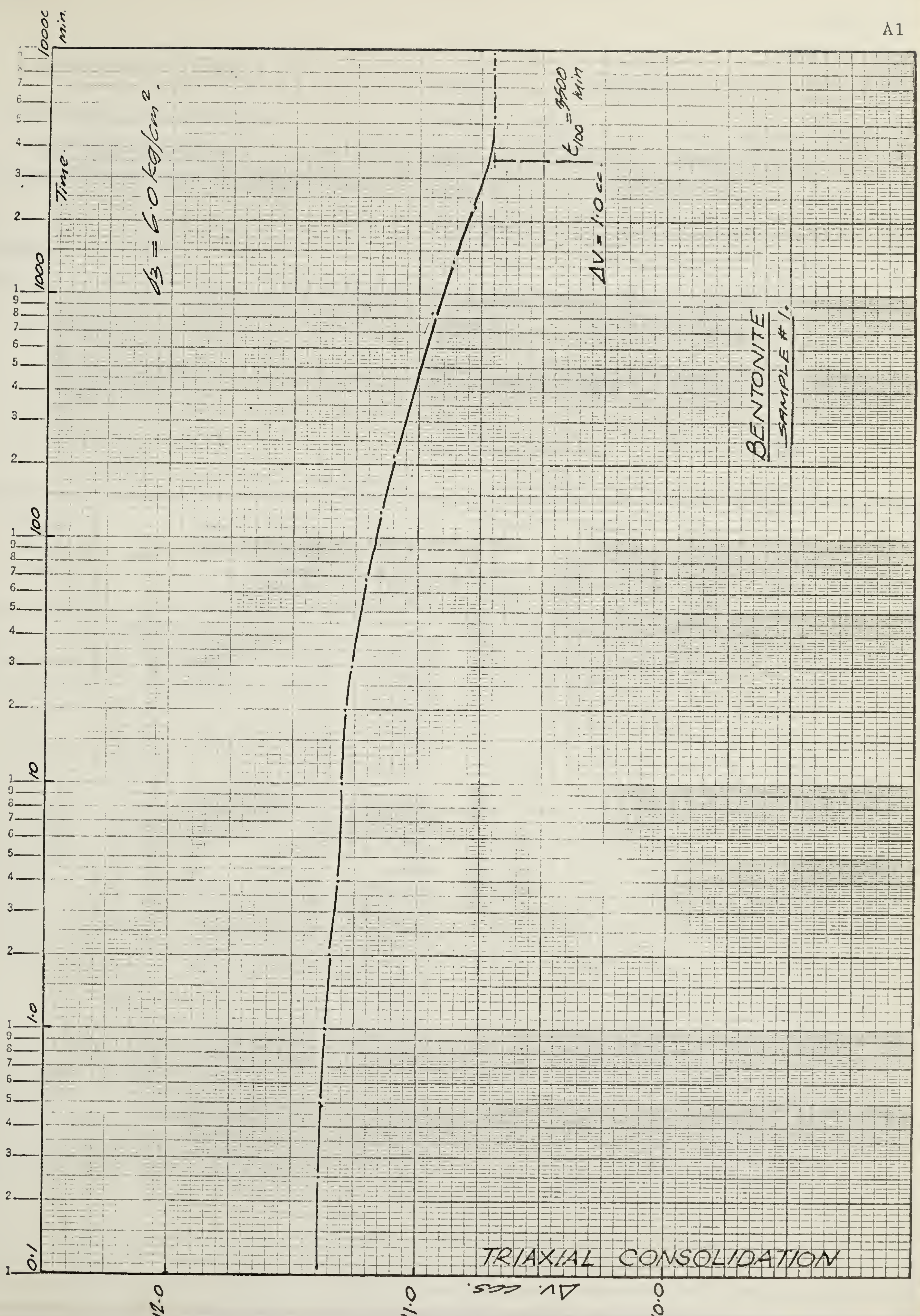
Terzaghi, K. and Peck, R.B., "Soil Mechanics in Engineering Practice", J. Wiley and Sons Inc., New York, 1948.

Thomson, S. "Effect of Salt Content and Adsorbed Cations on the Shear Strength of a Remoulded Highly Plastic Clay Soil", Unpublished Ph.D. Thesis, University of Alberta, Edmonton, 1963.

APPENDIX A

TYPICAL DATA SHEETS FOR

CONSOLIDATED UNDRAINED TRIAXIAL TESTS


TRIAXIAL CONSOLIDATION
BENTONITE
SAMPLE # 1.
$\sigma_3 = 6.0\ kg/cm^2$.
$\Delta V = 1.0$ cc.
$t_{100} = 3500$ min
Time.
10000 min.
1000
100
10
1.0
0.1
12.0
11.0
10.0
ΔV ccs.

Sheet 1

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

IAXIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS.

Hole No. 3

Depth 94'-6" Sample #1

Engineer W.T.PAINTER.

Technician

Date of Test JANUARY 1st 1965

Test Lateral Pressure σ_3 = 6·00 kg/cm².

Back Pressure = 2·00 kg/cm²

Remarks Consolidated Undrained Strain Rate – 1·25% per hour.

Area Correction Factor K = 1·008.

ta at Failure

$(\sigma_1 - \sigma_3)$ = 2·43 kg/cm² σ_1 = 8·43 kg/cm²

= 1·11 kg/cm²

1 = 7·32 kg/cm²

3 = 4·89 kg/cm²

ain 4·0 %

Initial Length = 8·13 cm. Initial C.S.A = 10·63 cm².

Time min	Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const δ kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm² Pp	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\frac{Pp}{\sigma_1 - \sigma_3}$
1/1/65.		10·63	—	—	—	-0·16	6·16	6·16	1·00	—	—
		10·65	66	·0465	·2905	+0·04	6·25	5·96	1·05	0·16	·137
	24·4	10·66	207	·0449	·8789	0·45	6·03	5·15	1·17	0·30	·510
	40·7	10·68	332·5	·0437	1·3714	0·68	6·69	5·32	1·26	0·50	·496
	57·7	10·71	411·6	·0431	1·6696	0·75	6·92	5·25	1·32	0·71	·449
1725	81·3	10·74	476·8	·0427	1·9108	0·82	7·09	5·18	1·37	1·00	·429
	101·6	10·76	511·6	·0425	2·0370	0·88	7·16	5·12	1·40	1·25	·432
	121·9	10·79	538·0	·0424	2·1310	0·90	7·23	5·10	1·42	1·50	·422
	178·8	10·86	586·1	·0422	2·2957	1·03	7·27	4·97	1·46	2·20	·449
	203·3	10·90	600·2	·0422	2·3423	1·05	7·29	4·95	1·47	2·50	·448
	227·6	10·94	611·1	·0421	2·3705	1·07	7·30	4·93	1·48	2·80	·451
	243·9	10·96	616·7	·0421	2·3878	1·07	7·32	4·93	1·48	3·00	·448
	264·2	10·99	623·0	·0421	2·4057	1·10	7·31	4·90	1·49	3·25	·457
	284·6	11·02	629·8	·0421	2·4252	1·10	7·33	4·90	1·50	3·50	·453
	304·8	11·04	632·5	·0421	2·4313	1·10	7·33	4·90	1·50	3·75	·452
	[illegible]·2	11·07	636·0	·0420	2·4323	1·11	7·32	4·89	1·50	4·00	·456
	[illegible]·5	11·10	637·7	·0420	2·4322	1·12	7·31	4·88	1·50	4·20	·460
137	365·9	11·13	640·1	·0420	2·4348	1·13	7·30	4·87	1·50	4·50	·464
	382·1	11·15	641·1	·0420	2·4342	1·15	7·28	4·85	1·50	4·70	·472
	411·4	11·20	642·0	·0420	2·4267	1·15	7·28	4·85	1·50	5·06	·474
	463·4	11·27	643·2	·0420	2·4162	1·20	7·22	4·80	1·50	5·70	·496
	487·8	11·31	643·3	·0420	2·4080	1·20	7·21	4·80	1·50	6·00	·498
	528·4	11·37	641·8	·0420	2·3897	1·22	7·17	4·78	1·50	6·50	·510
	5[illegible]·1	11·43	641·8	·0420	2·3772	1·26	7·12	4·74	1·50	7·00	·530

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

TRIAXIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. 3

Depth 94'-6" Sample #1

Area Correction Factor K = 1.008

Time min	Strain Dial Div.	A_c cm2	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm2 P_P	Effective Stress $\bar{\sigma}_1$ Major	$\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\frac{P_P}{\sigma_1 - \sigma_3}$

Moisture Content – Initial 45.96%

Final 45.54%

Degree of Saturation – Initial 96.5%

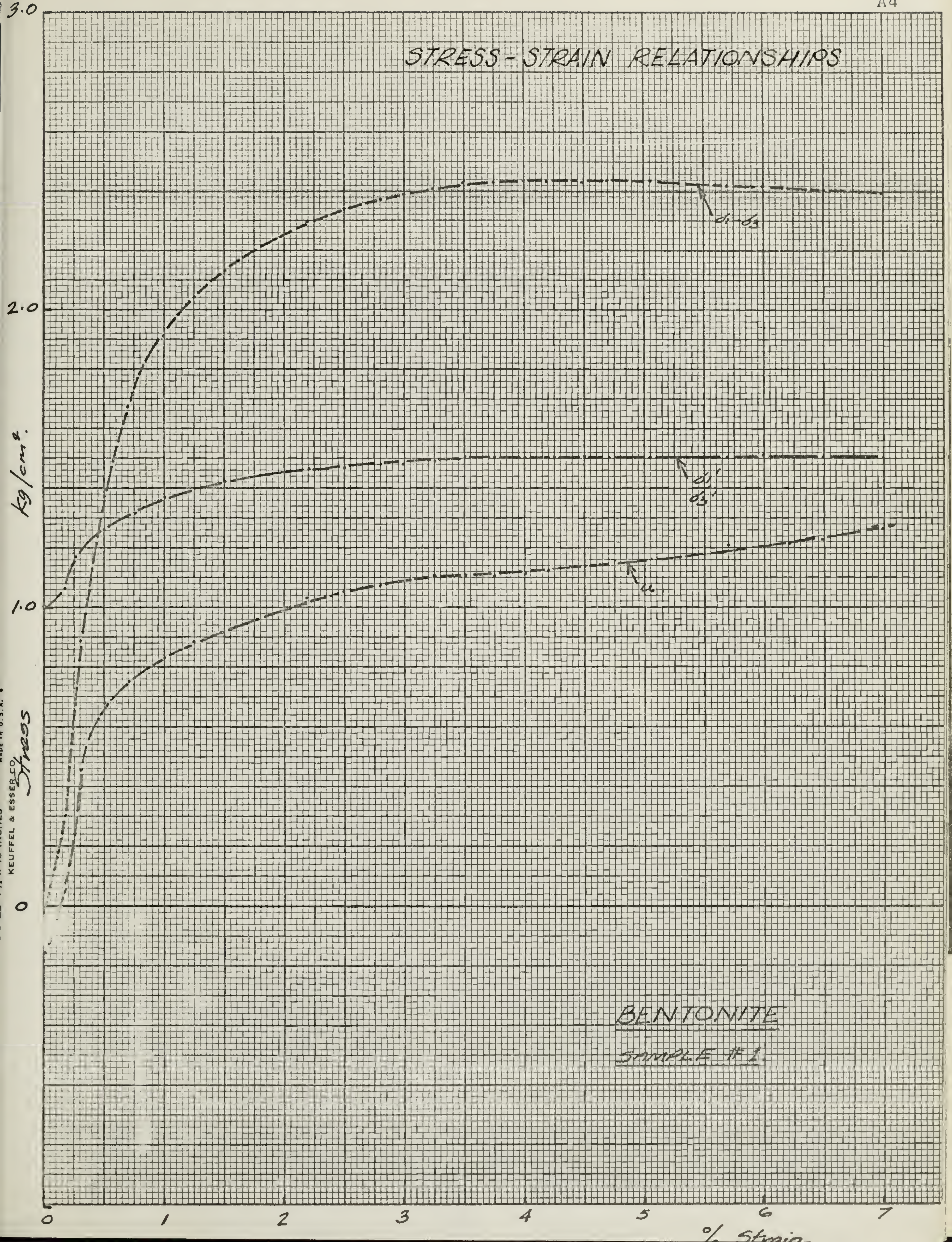
Final 96.1%

Void Ratio – Initial 1.26

Final 1.26

Pore Pressure Reaction 54%

Sketch of Failure


STRESS - STRAIN RELATIONSHIPS
3.0
2.0
1.0
0
Stress kg/cm².
σ1-σ3
σ1'/σ3'
u
0
1
2
3
4
5
6
7
% Strain
BENTONITE
SAMPLE #1
KEUFFEL & ESSER CO. MADE IN U.S.A.

Time. min.
σ3 = 8.0 kg/cm²
t100 = 620 min
ΔV = 0.50 cc.
DARK BROWN CLAY SHALE
SAMPLE #6.
TRIAXIAL CONSOLIDATION
0.1
1.0
10
100
1000
10000
19.0
18.0
ΔV: ccs.

Sheet 1 A6

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

TRIAXIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. 3

Depth 87'-0" Sample #6

Engineer W. T. PAINTER.

Technician — " —

Date of Test MARCH 9th 1965

Test Lateral Pressure σ_3 = 8.0 kg/cm²

Back Pressure = 1.0 kg/cm²

Remarks Consolidated Undrained at 1.25% per hour.

Area Correction Factor K = 1.004

Data at Failure

$(\sigma_1 - \sigma_3)$ = 6.21 kg/cm² σ_1 = 14.21 kg/cm²

P = 0.98

$\bar{\sigma}_1$ = 13.23 kg/cm²

$\bar{\sigma}_3$ = 7.02 kg/cm²

Strain 6.03 % %

Initial C.S.A. = 9.98

Initial Length 8.143

Time min	Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press. kg/cm² P_P	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\frac{P_P}{\sigma_1-\sigma_3}$
1145	–	9.98	–	–	–	-0.19	8.19	8.19	1.00	–	
	8.1	9.99	89.2	.0986	0.8839	-0.18	9.06	8.18	1.11	0.1	-.20
	20.37	10.00	193.0	.0918	1.7788	-0.17	9.95	8.17	1.22	0.25	-.10
	40.7	10.03	300.3	.0877	2.6362	-0.10	10.74	8.10	1.33	0.50	-.04
	61.1	10.06	378.0	.0862	3.2519	0.00	11.25	8.00	1.41	0.75	0
1339	81.4	10.08	431.0	.0857	3.6790	+0.08	11.60	7.92	1.47	1.00	.02
	101.8	10.10	478.0	.0853	4.0531	+0.16	11.89	7.84	1.52	1.25	.04
	122.1	10.13	515.5	.0851	4.3479	0.23	12.12	7.77	1.56	1.50	.05
	14[illegible].5	10.15	552.0	.0849	4.6356	0.30	12.34	7.70	1.61	1.75	.06
	170.0	10.18	591.2	.0847	4.9385	0.38	12.56	7.62	1.65	2.09	.08
	183.2	10.21	608.0	.0846	5.0580	0.43	12.63	7.57	1.67	2.25	.08
	203.6	10.24	632.0	.0845	5.2361	0.49	12.75	7.51	1.70	2.50	.09
	223.9	10.26	654.[illegible]	.0844	5.4031	0.54	12.89	7.46	1.73	2.75	.10
	244.3	10.29	671.9	.0844	5.5331	0.59	12.94	7.41	1.75	3.00	.11
	264.6	10.32	688.3	.0843	5.6449	0.64	13.00	7.36	1.77	3.25	.11
	287.0	10.34	704.2	.0842	5.7573	0.69	13.07	7.31	1.79	3.52	.12
	305.4	10.37	716.0	.0842	5.8369	0.73	13.11	7.27	1.80	3.75	.13
1655	325.7	10.40	728.0	.0841	5.9105	0.76	13.15	7.24	1.82	4.00	.13
	346.1	10.42	739.7	.0841	5.9940	0.80	13.19	7.20	1.83	4.25	.13
	366.4	10.45	749.2	.0840	6.0464	0.83	13.22	7.17	1.84	4.50	.14
	386.8	10.48	757.8	.0840	6.0983	0.88	13.22	7.12	1.86	4.75	.14
	407.2	10.51	764.7	.0840	6.1362	0.89	13.25	7.11	1.86	5.00	.15
	427.5	10.53	770.2	.0840	6.1686	0.91	13.26	7.09	1.87	5.25	.15
	[illegible]7.9	10.56	775.2	.0839	6.1837	0.93	13.25	7.07	1.87	5.50	.15
	471.0	10.59	780.0	.0839	6.2043	0.96	13.24	7.04	1.88	5.79	.15
	491.0	10.62	782.9	.0839	6.2098	0.98	13.23	7.02	1.88	6.03	.16
	508.9	10.65	785.0	.0839	6.2088	0.99	13.22	7.01	1.89	6.25	.16
	529.3	10.67	787.0	.0839	6.2131	0.99	13.22	7.01	1.89	6.50	.16
	620.0	10.80	790.2	.0839	6.1632	1.04	13.12	6.96	1.89	7.60	.17

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

TRIAXIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. #3

Depth 87'-0" Sample #6

Area Correction Factor $K = 1.004$

Time min	Strain Dial Div.	A_c cm^2	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm^2 P_p	Effective Stress Major $\bar{\sigma}_1$	Effective Stress Minor $\bar{\sigma}_3$	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\frac{P_p}{\sigma_1-\sigma_3}$

isture Content – Initial 34.90%

Final 34.23%

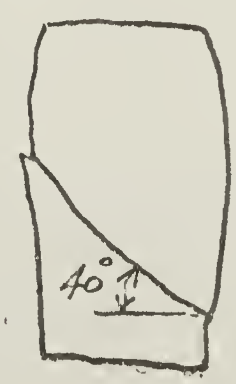
gree of Saturation – Initial 93.25%

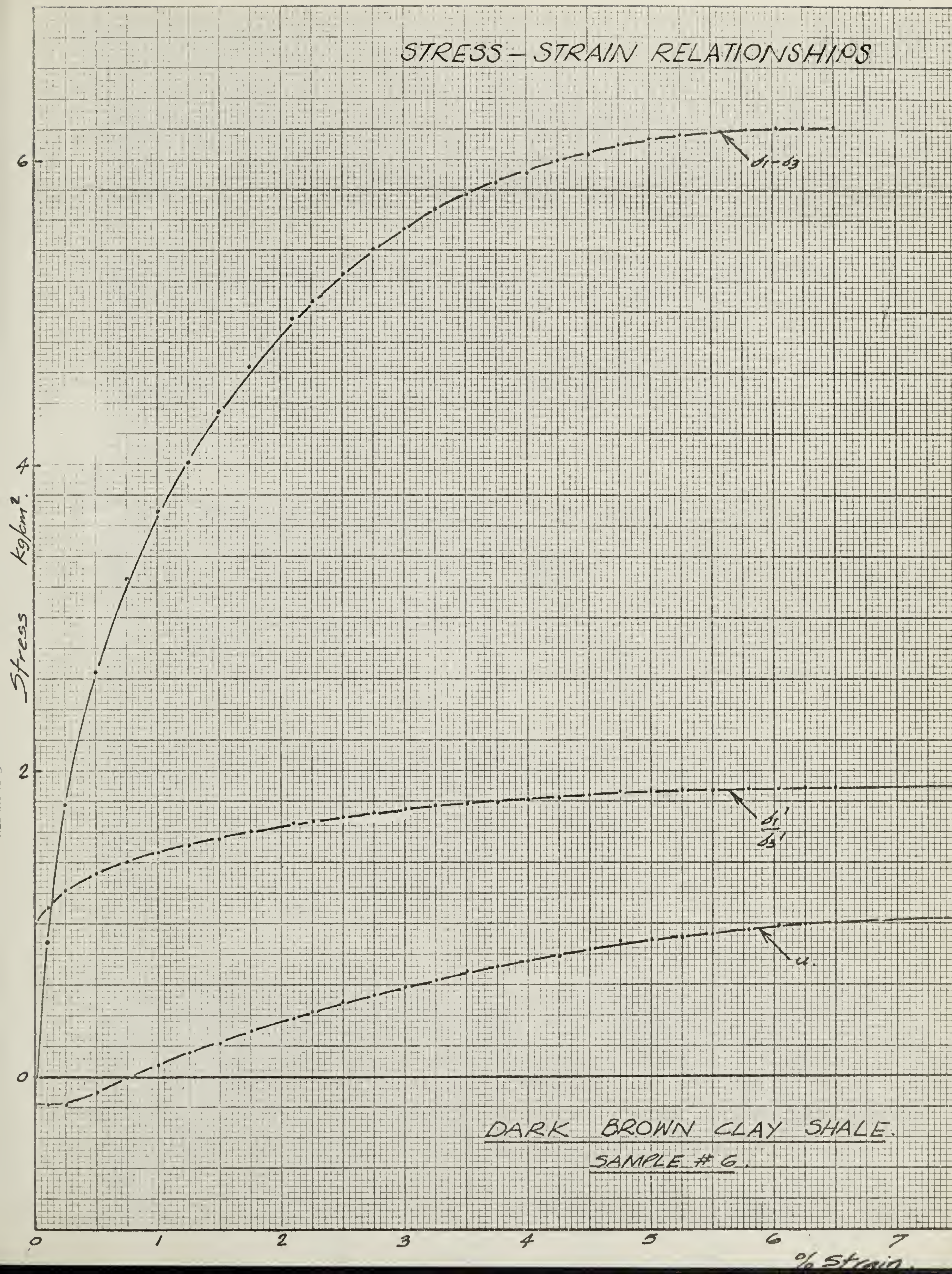
Final 96.80%

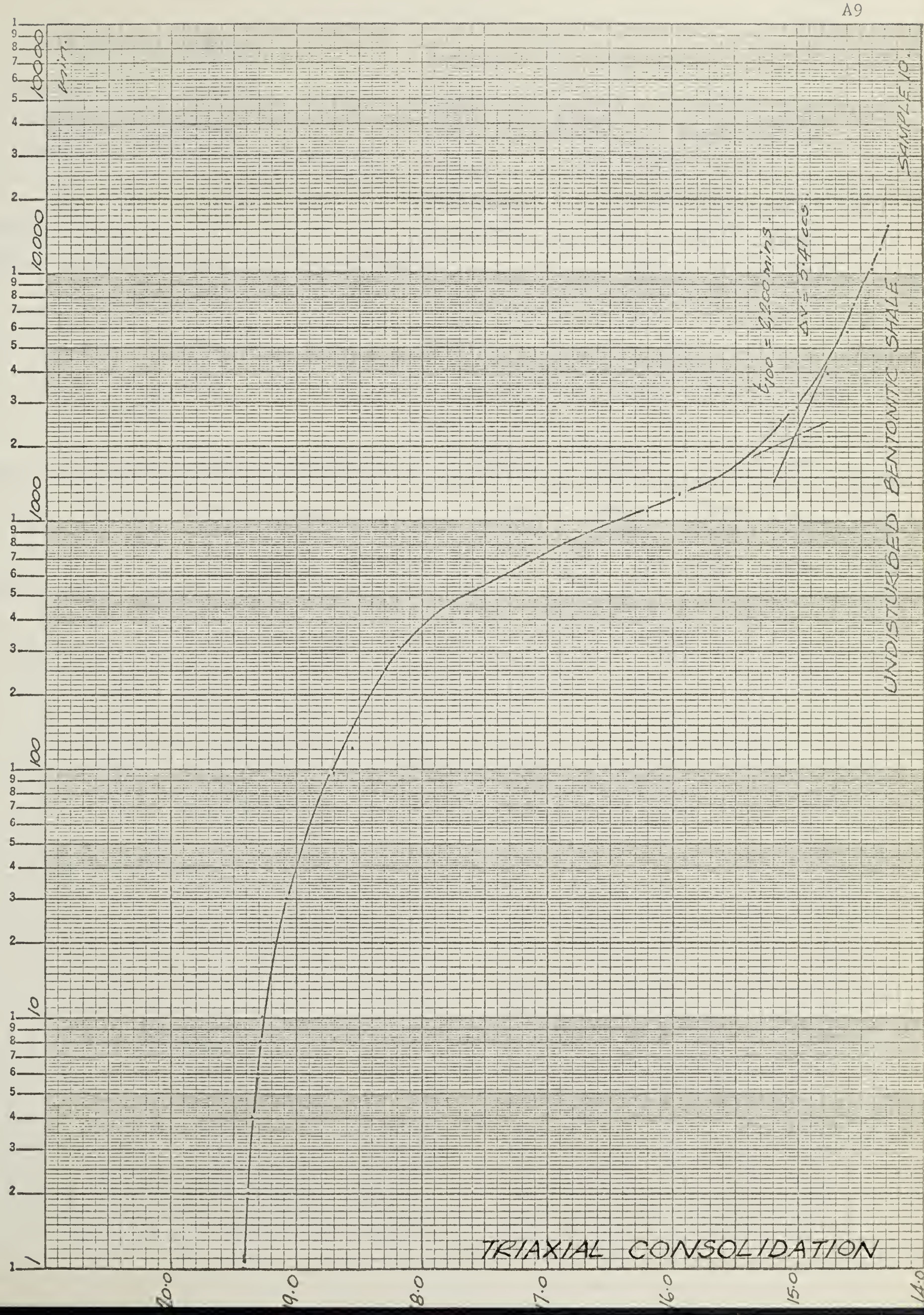
id Ratio – Initial 1.02

Final 0.96

Sketch of Failure




STRESS - STRAIN RELATIONSHIPS
Stress kg/cm².
6
4
2
0
σ1-σ3
σ1'/σ3'
u.
DARK BROWN CLAY SHALE.
SAMPLE # 6.
0
1
2
3
4
5
6
7
% Strain.


UNDISTURBED BENTONITIC SHALE
SAMPLE 10.
TRIAXIAL CONSOLIDATION
10,000
1000
100
10
1
20.0
19.0
18.0
17.0
16.0
15.0
14.0

UNIVERSITY OF ALBERTA

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Soil Mechanics Laboratory

RIAXIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. #6

Depth $2\frac{1}{2}$ ft. Sample #10

Engineer N.T. PAINTER.

Technician —— " —— -

ata at Failure

$\sigma_1 - \sigma_3) =$ 3.89 kg/cm² $\sigma_1 =$ 9.04 kg/cm²

= 1.86 kg/cm²

$\bar{\sigma}_1 =$ 7.18 kg/cm²

$\bar{\sigma}_3 =$ 3.29 kg/cm²

rain 5.75 %

Date of Test APRIL 29th 1965

Test Lateral Pressure σ_3 = 5.15 kg/cm²

Back Pressure σ_b = 1.0 kg/cm².

Remarks Consolidated Undrained at 1.25% per hour strain rate.

Area Correction Factor K = 1.046.

Initial csa = 10.04 cm² Initial Length 8.154 cms.

Time min	Strain Dial Div. 0	A_c cm²	No. of Stress Dial Div. +1	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm² Pp	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\bar{A}$ $\frac{P_p}{\sigma_1 - \sigma_3}$
1030	—	10.04	—	—	—	-0.18	5.33	5.33	1.00	—	
	8.2	10.05	55.2	0.0996	0.5722	-0.18	5.90	5.33	1.11	0.1	
	20.4	10.07	117.3	0.0974	1.1867	-0.12	6.46	5.27	1.23	0.25	
	40.8	10.09	183.2	0.0924	1.7548	+0.03	6.87	5.12	1.34	0.50	
	61.2	10.12	231.2	0.0900	2.1507	+0.17	7.13	4.98	1.43	0.75	
	81.5	10.14	267.9	0.0887	2.4513	+0.37	7.23	4.78	1.51	1.0	
1211	101.9	10.17	294.5	0.0879	2.6625	+0.53	7.28	4.62	1.57	1.25	
	126.0	10.20	322.5	0.0872	2.8839	0.73	7.30	4.42	1.65	1.55	
	142.7	10.22	339.2	0.0870	3.0203	0.86	7.31	4.29	1.70	1.75	
	163.1	10.24	359.0	0.0865	3.1721	0.97	7.35	4.18	1.76	2.00	
	183.5	10.27	373.0	0.0863	3.2785	1.06	7.37	4.09	1.80	2.25	
	203.9	10.30	386.4	0.0862	3.3825	1.21	7.32	3.94	1.86	2.50	
	224.2	10.32	397.8	0.0860	3.4675	1.30	7.32	3.85	1.90	2.75	
	244.6	10.35	407.8	0.0859	3.5402	1.40	7.29	3.75	1.94	3.00	
	265.0	10.38	417.0	0.0858	3.6054	1.48	7.28	3.67	1.98	3.25	
	285.4	10.40	425.5	0.0858	3.6719	1.54	7.28	3.61	2.02	3.50	
	311.0	10.43	[illegible]	0.0857	3.7275	1.63	7.25	3.52	2.06	3.81	
	330.0	10.46	[illegible]	0.0856	3.7596	1.72	7.19	3.43	2.10	4.05	
	346.5	10.49	443.2	0.0856	3.7829	1.75	7.18	3.40	2.11	4.25	
	366.9.	10.51	448.1	0.0855	3.8130	1.81	7.15	3.34	2.14	4.50	
	407.7	10.57	455.5	0.0855	3.8539	1.79	7.21	3.36	2.15	5.00	
	428.1	10.60	459.0	0.0855	3.8726	1.84	7.18	3.31	2.17	5.25	
	448.5	10.62	461.0	0.0855	3.8822	1.86	7.17	3.29	2.18	5.50	
	468.9	10.65	463.0	0.0855	3.8880	1.86	7.18	3.29	2.18	5.75	
	489.2	10.68	464.5	0.0854	3.8851	1.90	7.14	3.25	2.20	6.00	
100	509.6	10.71	466.6	0.0854	3.8917	1.89	7.15	3.26	2.19	6.25	

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

TRIAXIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. #6

Depth 2½-4. Sample #10

Area Correction Factor K = 1.046

Time min	Strain Dial Div.	A_c cm^2	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm^2 P_P	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\bar{A}$ $\frac{P_P}{\sigma_1 - \sigma_3}$

loisture Content – Initial 29.8%

Final 28.5%

egree of Saturation – Initial 87.3%

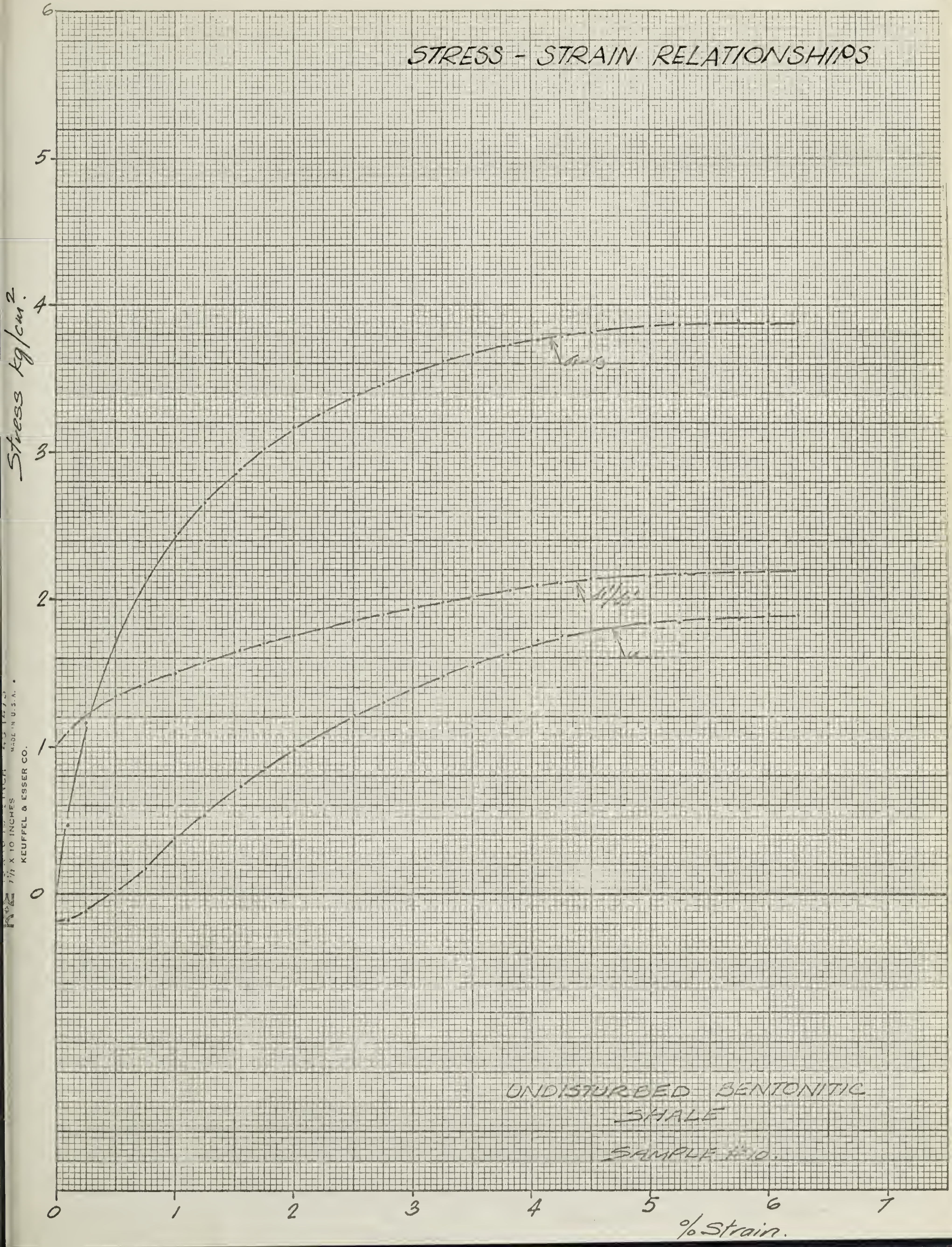
Final 93.7%

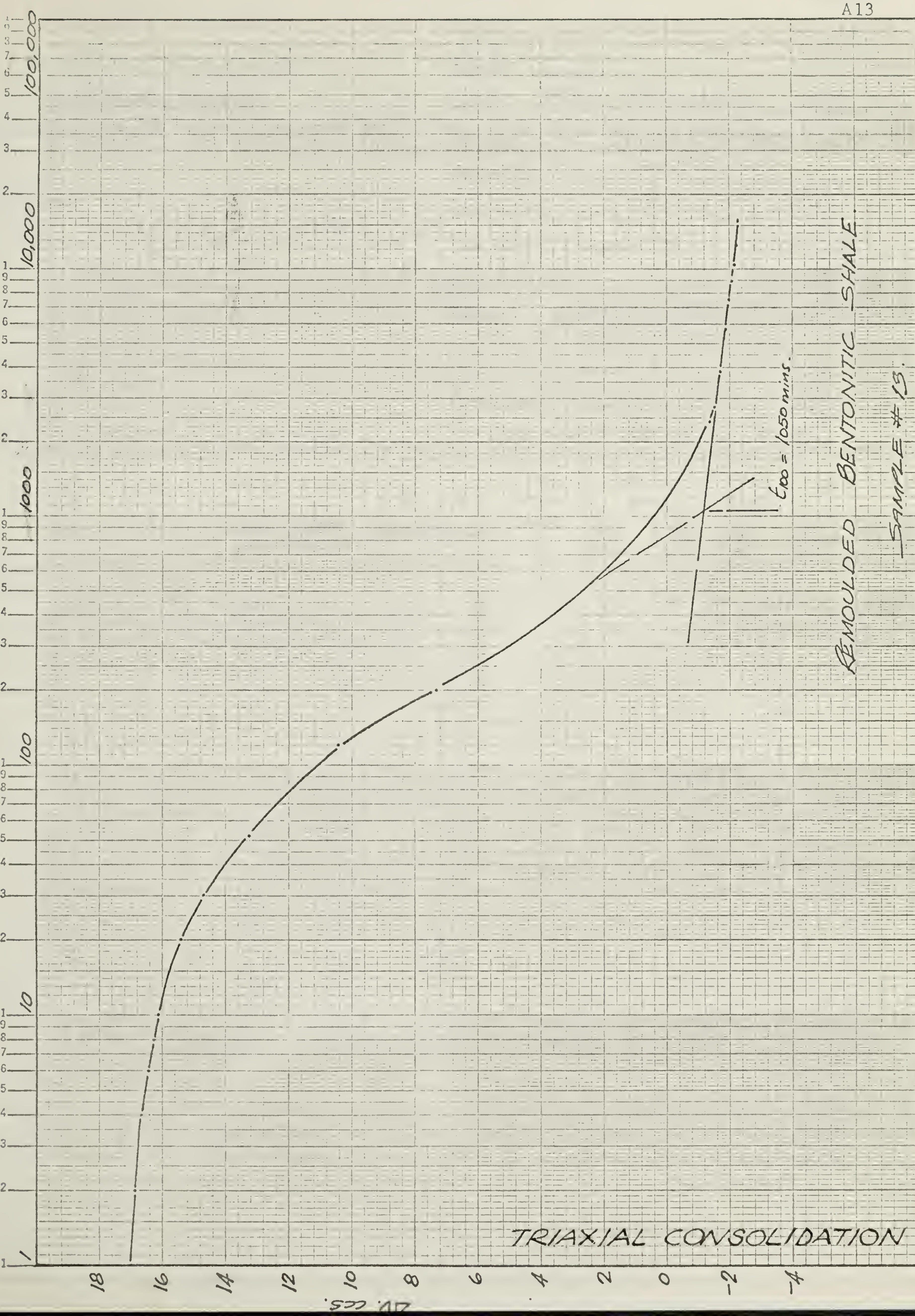
'oid Ratio – Initial .922

Final .821

ore Pressure Reaction 10%

Sketch of Failure


STRESS - STRAIN RELATIONSHIPS
Stress kg/cm.²
0
1
2
3
4
5
6
% Strain.
UNDISTURBED BENTONITIC
SHALE
SAMPLE #10.
KEUFFEL & ESSER CO.
MADE IN U.S.A.


REMOULDED BENTONITIC SHALE
SAMPLE #15.
t_{100} = 1050 mins.
TRIAXIAL CONSOLIDATION
100,000
10,000
1000
100
10
1
18
16
14
12
10
8
6
4
2
0
-2
-4
ΔV. ccs.

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

TRIAXIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. # 61

Depth 7½ - 17½' Sample # 13.

Engineer W. T. PAINTER.

Technician ——— , ———

Date of Test MAY 1st 1965.

Test Lateral Pressure σ_3 = 3·50 kg/cm²

Back Pressure = 1·00 kg/cm²

Remarks Consolidated-undrained @ 1·25% Strain/hour.

Area Correction Factor K = 1·260

Data at Failure

$(\sigma_1 - \sigma_3)$ = 2·19 kg/cm² σ_1 = 5·69 kg/cm²

P = 1·43 kg/cm²

$\bar{\sigma}_1$ = 4·26 kg/cm²

$\bar{\sigma}_3$ = 2·07 kg/cm²

Strain 3·50 %

Initial area = 9·91 Initial length = 8·15

Time min	Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press. kg/cm² P_p	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\bar{A} = \frac{P_p}{\sigma_1 - \sigma_3}$
1818	—	9·91	—	—	—	−0·01	3·51	3·51	1·00	—	—
	8·2	9·92	17·8	0·1005	0·2272	0·02	3·71	3·48	1·07	0·1	
	20·4	9·93	63·0	0·0995	0·7954	0·20	4·10	3·30	1·24	0·25	
	40·8	9·96	120·2	0·0972	1·4780	0·50	4·48	3·00	1·49	0·50	
	61·1	9·98	150·2	0·0950	1·8015	0·70	4·60	2·80	1·64	0·75	
	81·5	10·01	165·8	0·0937	1·9555	0·86	4·60	2·64	1·74	1·00	
	101·9	10·04	174·0	0·0931	2·0330	0·95	4·58	2·55	1·80	1·25	
2015	122·3	10·06	179·8	0·0928	2·0398	1·04	4·55	2·46	1·85	1·50	
	142·6	10·09	183·5	0·0926	2·1219	1·11	4·51	2·39	1·89	1·75	
2045	163·0	10·11	186·1	0·0924	2·1431	1·17	4·47	2·33	1·92	2·00	
	183·4	10·14	188·5	0·0921	2·1573	1·22	4·44	2·28	1·95	2·25	
	203·7	10·16	189·4	0·0921	2·1633	[illegible]	4·39	2·23	1·97	2·50	
	224·1	10·19	190·5	0·0920	2·1671	1·32	4·34	2·18	1·99	2·75	
	244·5	10·22	191·7	0·0920	2·1743	1·36	4·31	2·14	2·01	3·00	
	264·9	10·24	192·7	0·0919	2·1791	1·39	4·29	2·11	2·03	3·25	
	285·3	10·27	194·1	0·0918	2·1861	1·43	4·26	2·07	2·06	3·50	
	305·6	10·30	194·6	0·0918	2·1854	1·46	4·23	2·04	2·07	3·75	
2230	326·0	10·32	195·1	0·0917	2·1843	1·50	4·18	2·00	2·09	4·00	
	346·4	10·35	195·8	0·0917	2·1858	1·54	4·15	1·96	2·12	4·25	
	366·8	10·38	196·1	0·0917	2·1828	1·57	4·11	1·93	2·13	4·50	

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

TRIAXIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. # G1

Depth 7½ – 17½′ Sample # 13

Area Correction Factor $K = 1{\cdot}260$

Time min	Strain Dial Div.	A_c cm^2	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm^2 P_P	Effective Stress $\bar{\sigma}_1$ Major	$\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\bar{A}$ $\frac{P_P}{\sigma_1 - \sigma_3}$

oisture Content – Initial 46·9%

Final 55·2%

egree of Saturation – Initial 50·7%

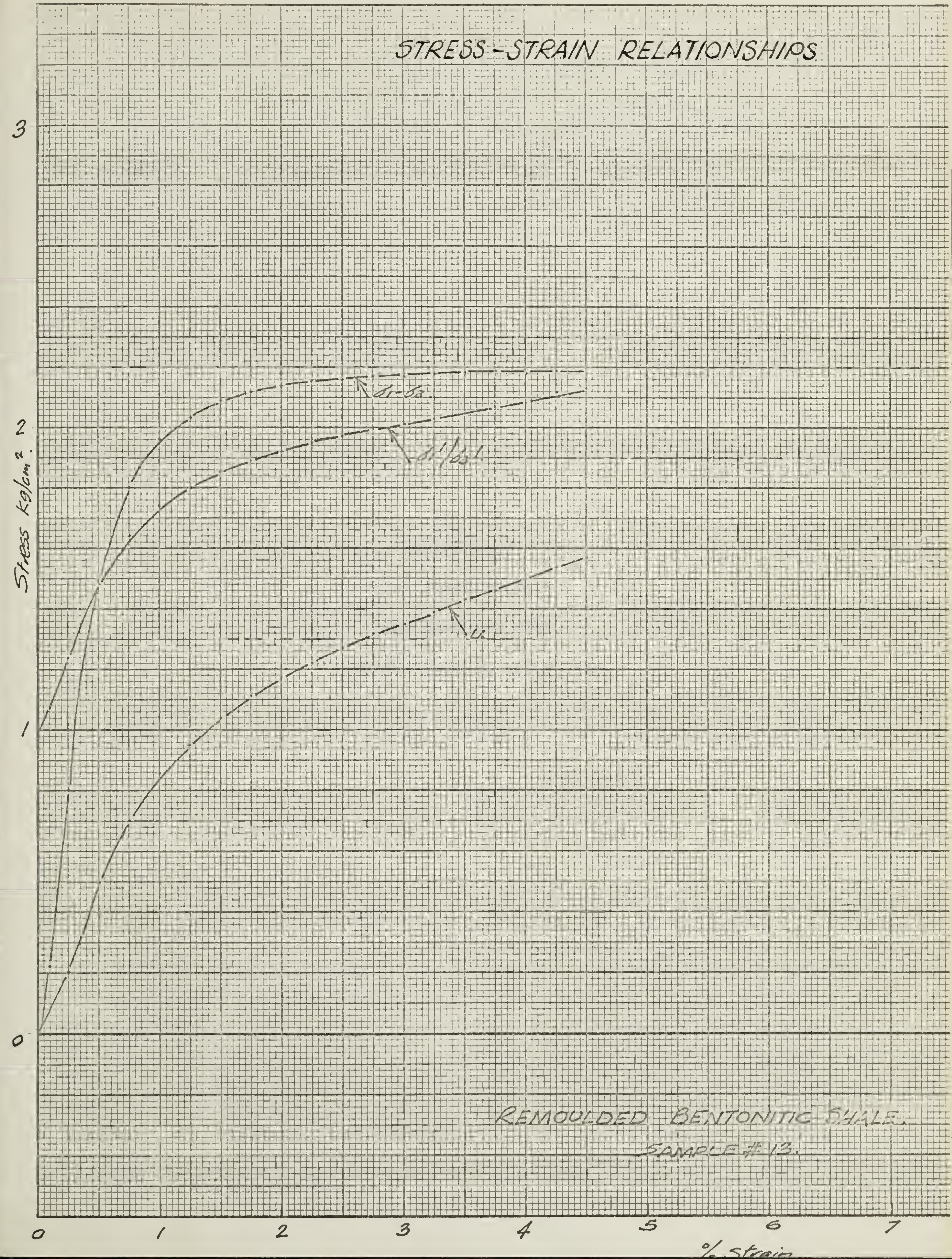
Final 96·7%

oid Ratio – Initial 2·50

Final 1·54

ore Pressure Reaction 72%

Sketch of Failure


STRESS-STRAIN RELATIONSHIPS
Stress kg/cm².
3
2
1
0
σ1-σ3
σ1'/σ3'
u
0
1
2
3
4
5
6
7
% strain
REMOULDED BENTONITIC SHALE.
SAMPLE # 13.

APPENDIX B

CLIMATALOGICAL DATA

FOR EDMONTON CITY

NOVEMBER 1964 to MARCH 1965

METEOROLOGICAL BRANCH - DEPARTMENT OF TRANSPORT - CANADA

MONTHLY METEOROLOGICAL SUMMARY
for
NOVEMBER 1964

A RECORD OF OBSERVATIONS TAKEN AT EDMONTON City, ALBERTA.

Date	TEMPERATURE (°F) Maximum	Minimum	Mean	Degree Days below 65°F	RELATIVE HUMIDITY (%) 0500 M.S.T.	1100 M.S.T.	1700 M.S.T.	2300 M.S.T.	WIND (m.p.h.) Average Speed from Anemometer	Prevailing Direction (by hours)	Maximum Speed for 1 hour with Direction	Duration of Bright Sunshine (hours)	Heat Units-Degree Days above 42°
1	55	33	44	21	50	65	66	71	13.1	NW	N 21	1.5	
2	39	30	35	30	77	67	75	97	3.8	S	N 8	0.0	
3	39	24	32	33	95	94	84	82	10.4	SE, S	SE,S,17	1.0	
4	44	31	38	27	76	79	80	80	7.0	NW	SE14	4.7	
5	48	32	40	25	87	50	38	55	8.0	SW	SW13	8.3	
6	56	32	44	21	48	36	38	49	11.7	SW	NW28	6.6	
7	42	25	34	31	66	48	54	92	7.6	SW	SW16	5.4	
8	39	18	29	36	91	78	50	74	5.7	S	S 12	8.4	
9	30	19	25	40	92	84	90	96	4.3	S	S 11	4.6	
10	30	21	26	39	99	97	91	97	4.6	NE	NE 9	0.0	
11	31	28	30	35	97	93	97	96	3.7	NW	NW 9	0.0	
12	30	26	28	37	97	93	88	98	2.8	NE	E 5	0.0	
13	29	18	24	41	91	93	89	89	11.0	NW	NW14	0.0	
14	19	6	13	52	87	88	76	80	6.6	S	NW11	5.9	
15	37	19	28	37	90	76	70	86	8.0	SW	S,SW10	5.3	
16	44	25	35	30	72	64	63	79	10.9	N	NW18	4.2	
17	34	19	27	38	91	82	90	91	3.7	S	NW,E,6	5.1	
18	28	18	23	42	87	89	85	88	3.5	W	S 7	0.0	
19	34	12	23	42	85	89	82	60	13.7	N,NE	NE20	0.0	
20	18	8	13	52	67	58	65	78	6.9	S	NE10	1.3	
21	45	17	31	34	86	64	62	83	6.3	SW	SW11	7.8	
22	27	13	20	45	76	82	94	92	6.8	NE	NE14	5.2	
23	25	12	19	46	89	95	93	96	9.3	SE	SE11	0.0	
24	22	0	11	54	93	79	72	74	16.8	NW	NW26	4.2	
25	1	-8	-4	69	77	77	79	79	7.3	E	NW12	3.1	
26	5	-2	2	63	79	77	77	76	11.0	E	E 15	0.0	
27	4	-1	2	63	78	74	76	72	9.2	E	E 13	0.0	
28	5	-7	-1	66	82	79	74	71	7.1	S	SE11	4.5	
29	-6	-17	-12	77	77	80	77	75	5.5	NE	NE11	1.3	
30	-3	-6	-5	70	80	75	71	77	11.5	NE	NE16	0.0	
31													
Mean	28.4	14.9	21.7	Total 1296	82	77	75	81	7.9	Prevailing SW	Maximum NW28	Total 88.5	
Normal	32.2	16.8	24.5	1220	-	-	-	-	8.2	S	29	101.5	

UDC:551.506.1(712.33)

PRECIPITATION (INCHES)					DAILY WEATHER SUMMARY FOR NOVEMBER 1964 at EDMONTON CITY.
Date	RAINFALL	ACTUAL DEPTH OF NEWLY FALLEN SNOW	TOTAL PRECIPITATION	TOTAL DEPTH OF NEW AND/OR OLD SNOW ON GROUND	
1	TR		TR		1. Cldy, warm, lgt rainshwr, wnds strong & gusty, bcmg lgt in aftn.
2	.01	.1	.02		2. Cldy, lgt wnds, lgt rain & snowshwr, cool.
3					3. Fog early morng, bcmg cldy with gusty wnds aftn & eveng, cool.
4	TR		TR		4. Cldy, lgt wnds, warm.
5					5. Sunny, lgt wnds.
6					6. Sunny with cldy periods, strong aftn. wnds.
7					7. Cldy bcmg sunny.
8					8. Sunny.
9					9. Cldy near noon. Aftn smoke. Fog late evng, lgt wnds
10	TR	1.3	.13		10. Fog ovrngt, snow in morng, fog & freezing drizzle in evng, lgt wnds.
11	TR	1.5	.15		11. Ovc fog, freezing drizzle, morng lgt snow, rest of day lgt wnds.
12		4.8	.39	6	12. Lgt snow all day, fog late evng, cool, lgt wnds.
13		1.0	.06	7	13. Lgt snow all day. Fog early morng, drftg snow in pm.
14		0.1	.01	8	14. Lgt snow, drftg snow early morng, mainly sunny & cool, during day lgt wnds.
15		0.7	.03	8	15. Lgt snow after midnight. Mostly sunny during day. Cool, Lgt wnds.
16				7	16. Cldy aftn & evng, mild, wnds ocnly gusty in aftn.
17				5	17. Mainly cldy, lgt wnds, fog late evng.
18		TR	TR	4	18. Cldy, foggy, snow early aftn, lgt wnds.
19	TR	TR	TR	4	19. Cldy, gusty wnds, lgt snow & freezing drizzle.
20				4	20. Cldy, cool, lgt wnds.
21				4	21. Few clds, lgt wnds, warm.
22				3	22. Clear in morng, cldy with fog aftn & evng. Cool, lgt wnds.
23		TR	TR	3	23. Ovc with fog most of day. Cool. Lgt wnds.
24		2.0	.15	3	24, Snow, fog in a.m. Temps. falling all day. Mainly strong wnds with drftg snow.
25		TR	TR	4	25. Cldy & cool, lgt snow in morng.
26		1.7	.10	4	26. Cldy, cold, lgt snow, lgt wnds.
27		1.6	.07	7	27. Cldy, cold, lgt snow, lgt wnds.
28		0.7	.04	7	28. Cldy clearing in evng, snow till noon. Cold, lgt wnds.
29		0.2	.01	7	29. Clr clding ovr in morng with lgt snow.
30		4.6	.30	8	30. Ovc with snow, wndy.
31					
Total	.01	20.3	1.46		
Normal	.14	7.4	.88		

THUNDERSTORMS

NO.	NORMAL	DATES
0	0	-

TOTAL PRECIPITATION; NO. OF DAYS WITH:					SNOWFALL; NO. OF DAYS WITH:				
0.01" or more	0.04" or more	0.25" or more	0.50" or more	1.00" or more	0.1" or more	0.4" or more	2.5" or more	5.0" or more	10.0" or more
13	9	2	0	0	13	10	2	0	0

(1) T = Trace = negligible amount of rain or snow. Precipitation data are based on 24-hour period beginning 11 p.m. and ending 11 p.m. the following day.

(3) Measurement taken at 0500 M.S.T., 1100 M.S.T., 1700 M.S.T. and 2300 M.S.T.

METEOROLOGICAL BRANCH - DEPARTMENT OF TRANSPORT - CANADA

MONTHLY METEOROLOGICAL SUMMARY

for

DECEMBER 1964

A RECORD OF OBSERVATIONS TAKEN AT EDMONTON, ALBERTA. (City)

Date	TEMPERATURE (°F)			Degree Days below 65°F	RELATIVE HUMIDITY (%)				WIND (m.p.h.)			Duration of Bright Sunshine (hours)	Heat Units-Degree Days above 42°
	Maximum	Minimum	Mean		0500 M.S.T.	1100 M.S.T.	1700 M.S.T.	2300 M.S.T.	Average Speed from Anemometer	Prevailing Direction (by hours)	Maximum Speed for 1 hour with Direction		
1	-1	-5	-3	68	77	69	71	68	11.2	SE	SE 14	0.0	
2	6	-2	2	63	74	74	80	75	13.1	SE	SE 16	0.0	
3	14	6	10	55	81	79	74	84	8.1	S	S 13	0.0	
4	15	9	12	53	89	90	85	94	4.8	SW	SW 7	4.4	
5	27	8	18	47	90	84	77	78	6.1	SW	SW 8	4.8	
6	33	17	25	40	81	71	65	76	7.1	SW	SW 10	6.4	
7	31	14	23	42	75	62	60	63	7.0	SW	SW 11	5.8	
8	37	20	29	36	77	68	72	61	7.0	SW	SW 10	6.1	
9	32	21	27	38	73	62	69	86	7.0	SW	S,SW 9	5.5	
10	24	10	17	48	87	86	90	95	6.0	NW	S 10	3.8	
11	19	2	11	54	93	88	88	85	10.0	NW	NW 14	1.7	
12	4	-10	-3	68	82	81	81	79	3.4	E	E 6	2.8	
13	3	-18	-8	73	79	71	72	76	10.1	N	NW 19	0.0	
14	- 8	-24	-16	81	75	72	71	74	12.2	E	NW 27	2.2	
15	-23	-31	-27	92	72	72	54	55	24.4	NW	NW 29	2.4	
16	-17	-28	-23	88	60	68	55	64	10.6	S,NW	NW 21	5.9	
17	5	-21	- 8	73	54	60	76	76	6.7	SW	SW 13	0.4	
18	- 3	-12	- 8	73	79	71	70	71	9.0	NE	NE 14	0.0	
19	-12	-20	-16	81	64	76	70	70	7.9	NE	NE 12	2.3	
20	-15	-20	-18	83	76	57	63	76	8.0	N,NW	NE 12	0.7	
21	-14	-22	-18	83	64	65	72	75	3.3	SW	S 7	0.0	
22	-13	-24	-19	84	87	75	82	78	4.5	N	NW 10	0.0	
23	-11	-20	-16	81	78	64	78	75	8.7	NW	N 15	3.1	
24	-10	-24	-17	82	81	82	62	70	5.0	S	SW,S 7	2.4	
25	9	-17	- 4	69	82	61	65	79	6.6	SW	W 12	4.[illegible]	
26	5	-10	- 3	68	80	69	73	73	11.5	SE	SE 19	0.7	
27	0	- 5	- 3	68	78	69	79	79	7.7	NW	E 13	0.0	
28	- 5	-16	-11	76	75	72	75	68	4.0	NE	NW 9	0.0	
29	-12	-20	-16	81	71	74	78	87	4.2	NE	NE 10	0.0	
30	-12	-22	-17	82	68	81	73	71	5.0	NE	S 12	0.0	
31	-12	-24	-18	83	81	68	72	76	6.7	S	S 13	5.6	
Mean	3.1	-9.3	-3.1	Total 2113	77	72	73	75	8.0	Prevailing SW	Maximum NW 29	Total 71.3	
Normal	21.0	5.5	13.3	1660					7.7	S	30	77.8	

UDC:551.506.1(712.33)

PRECIPITATION (INCHES)				
Date	RAINFALL	ACTUAL DEPTH OF NEWLY FALLEN SNOW	TOTAL PRECIPITATION	TOTAL DEPTH OF NEW AND/OR OLD SNOW ON GROUND
1		1.9	.12	11
2		.2	.01	11
3		.3	.02	11
4		TR	TR	11
5				10
6				10
7				9
8				9
9				9
10	TR		TR	9
11	TR	.1	.01	9
12		TR	TR	9
13		.1	.01	9
14		.2	.01	9
15		1.4	.08	9
16		.2	.01	9
17				9
18	-	.6	.04	9
19		.8	.05	9
20		.9	.06	10
21		TR	TR	10
22		1.1	.11	10
23		1.0	.07	12
24		TR	TR	11
25				11
26		TR	TR	11
27		.3	.02	11
28		1.0	.07	11
29		.3	.01	11
30		.1	.01	11
31		TR	TR	11
Total	TR	10.5	.71	
Normal	.05	9.4	.99	

THUNDERSTORMS		
NO.	NORMAL	DATES
0	0	-

DAILY WEATHER SUMMARY FOR DECEMBER 1964 AT EDMONTON CITY.

1. Ovc with snw.
2. Ovc with snw. Strong winds.
3. Ovc with snw. Lgt winds.
4. Cldy, clrng by eve. Lgt winds. Fog or smoke from late mrng.
5. Cldy in daytime. Lgt. winds. Fog clrd ovrngt.
6. Aftn cldiness. Lgt winds. Mild.
7. Mostly clr bcmg cldy in aftn. Mild. Lgt winds.
8. Few clds. Lgt winds. Mild.
9. Cldy. Lgt winds. Little Cooler.
10. Cldy during day. Cooler. Lgt. wnds. Ocnl.fog and smk in evng.
11. Lgt. frz. drizzle, lgt snw, fog, clrg late evng. Cold Mainly lgt wnds.
12. Clear in night. Cldy and colder with fog during day. Lgt wnds.
13. Cldy during day. Very cold. Strong, gsty wnds in aftn with lgt snw and drftg snw.
14. Clr, Cldy in aftn. Extremely cold. Strong winds in evng. Lgt snow and blowing snw.
15. Cldy, snow & blowing snow, strong gusty winds becmg lgt in mrng. Exceptionally cold.
16. Extremely cold, mainly clr. Snow. Strong winds.
17. Cldy and cold, lgt winds.
18. Ovc with snw, cold, lgt winds.
19. Cldy with snw and fog, colder, light winds.
20. Cldy, clrg in evng. Lgt snw during night. Very cold. Lgt winds.
21. Cldy during day. Fog. Very cold. Lgt winds.
22. Cldy during day. Fog. Light snw in evng. Very cold. Lgt winds.
23. Cldy with snw erly mrng, clrg by eveng, very cold. Winds lgt.
24. Fog - lifting by noon, cldy clrng by eveng, very cold, lgt winds.
25. Vrbl cldnss, not so cold, lgt winds.
26. Vrbl cldnss, cold, winds lgt excpt ocnl gusty in aftn.
27. Overcast lgt snow, clrg late eve, lgt winds, cool.
28. Clear overnight. Cldy with snow during day, clrg late afternoon, lgt winds, cold.
29. Mainly clr overnight. Cldy with snw and fog during day. Fog, smk in evng. Cold. Lgt winds.
30. Clear overnight. Cldy with snw and fog during day. Cold. Lgt Winds.
31. Mainly clear. Fog during mrng. Cold. Lgt winds.

TOTAL PRECIPITATION; NO. OF DAYS WITH:					SNOWFALL; NO. OF DAYS WITH:				
0.01" or more	0.04" or more	0.25" or more	0.50" or more	1.00" or more	0.1" or more	0.4" or more	2.5" or more	5.0" or more	10.0" or more
17	8	0	0	0	17	8	0	0	0

(1) T = Trace = negligible amount of rain or snow. Precipitation data are based on 24-hour period beginning 11 p.m. and ending 11 p.m. the following day.

(3) Measurement taken at 0500 M.S.T., 1100 M.S.T., 1700 M.S.T. and 2300 M.S.T.

METEOROLOGICAL BRANCH - DEPARTMENT OF TRANSPORT - CANADA

MONTHLY METEOROLOGICAL SUMMARY

for

JANUARY 1965

A RECORD OF OBSERVATIONS TAKEN AT EDMONTON City, ALBERTA

Date	TEMPERATURE (°F)			Degree Days below 65°F	RELATIVE HUMIDITY (%)				WIND (m.p.h.)			Duration of Bright Sunshine (hours)	Heat Units-Degree Days above 42°
	Maximum	Minimum	Mean		0500 M.S.T.	1100 M.S.T.	1700 M.S.T.	2300 M.S.T.	Average Speed from Anemometer	Prevailing Direction (by hours)	Maximum Speed for 1 hour with Direction		
1	27	-20	4	61	82	78	65	71	9.6	NW	SW,NW18	0.0	
2	-12	-22	-17	82	71	55	55	62	12.2	N	N 18	0.0	
3	-22	-24	-23	88	74	74	61	54	10.3	N	NW13	0.0	
4	-23	-30	-27	92	57	63	58	54	5.9	W	NW13	6.5	
5	-23	-33	-28	93	71	72	81	88	2.6	NE	NE 7	0.0	
6	-21	-25	-23	88	81	54	75	74	5.4	N	NW 9	2.0	
7	-18	-26	-22	87	68	58	76	65	5.3	NW	NW10	0.0	
8	-20	-27	-24	89	73	58	57	70	4.7	W	N 11	0.4	
9	-10	-20	-15	80	62	70	69	82	5.3	S	S 11	0.0	
10	20	-19	1	64	77	79	83	75	8.0	S	N 15	4.4	
11	-7	-12	-10	75	74	68	75	79	5.3	NE	NE 9	0.0	
12	5	-22	-9	74	87	74	78	82	4.3	E,SE	SE 8	3.9	
13	39	5	22	43	86	73	61	74	7.9	SW	W 14	3.7	
14	42	34	38	27	97	88	77	87	7.3	SW	SW12	0.3	
15	45	28	37	28	94	79	64	69	6.8	SW	SW12	7.5	
16	43	28	36	29	72	73	66	81	5.9	SW	SW 9	5.8	
17	34	21	28	37	85	76	77	84	5.9	SW	S,SW 8	7.9	
18	38	20	29	36	82	81	73	94	4.0	SW	SW 8	5.3	
19	33	12	23	42	92	77	77	84	7.0	S	S 14	6.3	
20	42	23	33	32	75	64	76	89	10.3	NW	NW23	6.1	
21	34	13	24	41	94	87	86	84	10.9	E	E 19	0.0	
22	14	3	9	56	76	80	77	85	7.7	SE	SE 14	1.4	
23	25	2	14	51	88	68	58	74	9.2	SW	N 17	4.6	
24	8	-3	3	62	80	81	71	77	7.5	N	N 12	0.0	
25	18	-4	7	58	75	70	72	75	6.1	SW	SW 9	1.5	
26	17	-5	6	59	83	76	75	79	10.6	NE	NE20	0.0	
27	0	-8	-4	69	81	67	72	72	6.9	S	E,S 9	0.4	
28	8	0	4	61	76	76	71	78	6.3	E	SE11	0.5	
29	7	-1	3	62	76	75	74	76	6.9	N	N 14	3.6	
30	14	-2	6	59	79	79	80	79	11.0	W	NW25	0.0	
31	0	-12	-6	71	75	62	69	71	6.3	N	N 10	5.0	
Mean	11.5	-4.1	3.7	Total 1896	79	72	71	76	7.2	Prevailing SW	Maximum NW25	Total 77.1	
Normal	15.2	-2.0	6.6	1780	-	-	-	-	7.8	S	29	86.2	

UDC:551.506.1(712.33)

PRECIPITATION (INCHES)				
Date	RAINFALL	ACTUAL DEPTH OF NEWLY FALLEN SNOW	TOTAL PRECIPITATION	TOTAL DEPTH OF NEW AND/OR OLD SNOW ON GROUND
1		TR	TR	11
2		TR	TR	11
3		1.3	.06	11
4		TR	TR	12
5		.3	.03	12
6		1.1	.07	12
7		.7	.06	12
8		.7	.03	13
9		1.5	.08	13
10		.4	.02	14
11		1.2	.07	14
12		1.2	.05	15
13		2.4	.18	18
14	.37		.37	16
15				11
16				10
17				10
18				9
19				8
20				8
21	.02	2.1	.17	8
22		1.1	.07	10
23		TR	TR	10
24		.8	.06	9
25		TR	TR	9
26		1.1	.05	9
27		1.4	.09	11
28		2.0	.12	11
29		.1	TR	13
30		4.7	.38	13
31		.5	.04	17
Total	.39	24.6	2.00	
Normal	.01	9.4	.95	

THUNDERSTORMS		
NO.	NORMAL	DATES
0	0	-

DAILY WEATHER SUMMARY FOR JANUARY 1965 AT EDMONTON City

1. Mainly cldy. Warm aftn. Strong gusty wnd in eveng.
2. Cldy from erly morng. Cold. Mainly lgt wnds.
3. Cldy. Lgt snw. Mainly lgt wnds. Very cold.
4. Cldy, bcmg clear by eveng. Lgt wnds. Snw early morng.
5. Cold. Lgt wnds. Fog & smoke. Lgt snow in eveng.
6. Cldy. Lgt snw morng & evng. Lgt wnds. Cold.
7. Cldy with lgt snw & fog during day. Cold. Lgt wnds.
8. Cldy. Lgt snw morng & evng. Very cold. Lgt wnds.
9. Cldy with lgt snw. Clearing late evng. Cold. Lgt wnds.
10. Cldy. Fog in early morng. Snw late evng. Moderate wnds.
11. Snw all day. Cold. Lgt wnds.
12. Cldy. Cold. Fog in aftn. Snw in evng. Lgt wnds.
13. Cldy. Lgt snw during night. Lgt wnds. Milder.
14. Cldy & mild. Lgt rain during night & early morng.
15. Cldy in daytime. Mild. Lgt wnds.
16. Mostly cldy & mild. Lgt wnds.
17. A few clds. Mild. Lgt wnds.
18. Cldy clearing in evng. Mild. Lgt wnds.
19. Partly cldy. Not quite so mild. Lgt wnds.
20. Partly cldy. Mild. Lgt wnds,with brief gusts.
21. Cldy with snw or freezing rain, vrbl wnds. Mild.
22. Cldy with snw clearing in aftn. Lgt wnds.
23. Vrbl cldnss, lgt snw in evng, lgt wnds. Mild.
24. Cldy, cold. Snw all day. Wnds lgt.
25. Cldy & cold. Lgt wnds. Lgt snw late evng.
26. Cldy with lgt snw. Cold. Lgt to mdt wnds. Drftg
27. Cldy & cold. Lgt snw all day. Lgt wnds.
28. Cldy with snw. Lgt E wnds.
29. Cldy, snw endg by noon, lgt N wnds.
30. Clear, bcmg cldy with snw in evng, lgt wnds bcmg NE 26 gusty to 34 in evng.
31. Vrbl cldnss, snw endg in morng, lgt wnds, turning colder.

TOTAL PRECIPITATION; NO. OF DAYS WITH:					SNOWFALL; NO. OF DAYS WITH:				
0.01" or more	0.04" or more	0.25" or more	0.50" or more	1.00" or more	0.1" or more	0.4" or more	2.5" or more	5.0" or more	10.0" or more
19	16	2	0	0	19	17	1	0	0

(1) T = Trace = negligible amount of rain or snow. Precipitation data are based on 24-hour period beginning 11 p.m. and ending 11 p.m. the following day.

(2) [struck out]

(3) Measurement taken at 0500 M.S.T., 1100 M.S.T., 1700 M.S.T. and 2300 M.S.T.

METEOROLOGICAL BRANCH - DEPARTMENT OF TRANSPORT - CANADA

MONTHLY METEOROLOGICAL SUMMARY

for

F E B R U A R Y 1965

A RECORD OF OBSERVATIONS TAKEN AT EDMONTON City, ALBERTA.

Date	TEMPERATURE (°F)			Degree Days below 65°F	RELATIVE HUMIDITY (%)				WIND (m.p.h.)			Duration of Bright Sunshine (hours)	Heat Units-Degree Days above 42°
	Maximum	Minimum	Mean		0500 M.S.T.	1100 M.S.T.	1700 M.S.T.	2300 M.S.T.	Average Speed from Anemometer	Prevailing Direction (by hours)	Maximum Speed for 1 hour with Direction		
1	0	-17	- 9	74	82	72	64	72	9.1	NW	NW 17	2.6	
2	3	-17	- 7	72	77	72	72	68	7.7	W	NW 12	6.7	
3	4	-22	- 9	74	81	68	68	67	8.2	S	SE 15	3.0	
4	12	2	7	58	78	78	72	79	5.6	N	S 10	0.0	
5	10	- 4	3	62	75	82	84	77	13.6	NE	NE 22	0.0	
6	- 4	-19	-12	77	80	62	58	82	10.7	NW	N,NW 20	5.9	
7	41	-17	12	53	69	85	90	64	7.4	S	SW 12	0.0	
8	39	12	26	39	94	85	72	82	9.2	SW	N 21	0.5	
9	12	-10	1	64	87	65	69	78	8.3	S	N 19	6.8	
10	21	- 2	10	55	81	82	73	72	5.0	SW	NW 10	2.3	
11	31	13	22	43	69	82	83	78	13.3	NW	NW 26	3.2	
12	25	4	15	50	84	89	75	71	10.6	S	S 17	7.9	
13	37	22	30	35	52	62	59	70	15.3	NW	NW 29	7.6	
14	25	- 9	8	57	74	61	63	68	10.3	N	N 25	4.3	
15	29	- 7	11	54	66	83	79	85	8.0	S	S 14	1.6	
16	38	11	25	40	85	68	79	76	8.8	SW	N 17	1.8	
17	38	10	24	41	84	83	75	82	7.3	SW	S,E 11	0.0	
18	46	19	33	32	96	90	80	68	7.0	NE	S,SW 12	0.0	
19	42	- 6	18	47	73	82	84	66	15.9	N	N 32	0.0	
20	- 3	-13	- 8	73	56	65	57	56	11.9	NW	NW 22	9.3	
21	- 1	-20	-11	76	77	53	74	74	9.5	SE	SE 17	0.0	
22	0	-12	- 6	71	71	71	65	69	6.6	S	E,S 11	6.7	
23	26	-12	7	58	69	60	73	75	9.1	S	N 14	2.7	
24	35	3	19	46	83	63	61	73	6.3	SW,S	S 10	2.3	
25	40	17	29	36	73	70	63	57	6.0	S	S 10	1.6	
26	39	16	28	37	72	80	72	78	7.9	N	N 16	0.0	
27	16	1	9	56	77	85	78	78	15.5	NE	NE 23	0.0	
28	4	- 7	- 2	67	74	74	64	71	9.3	N	NW 15	7.5	
29													
30													
31													
Mean	21.6	-2.3	9.7	Total 1547	76	74	72	73	9.4	Prevailing S	Maximum N 32	Total 84.3	
Normal	20.3	2.0	11.2	1520	-	-	-	-	7.9	S	28	114.4	

UDC:551.506.1(712.33)

PRECIPITATION (INCHES)				
Date	RAINFALL	ACTUAL DEPTH OF NEWLY FALLEN SNOW	TOTAL PRECIPITATION	TOTAL DEPTH OF NEW AND/OR OLD SNOW ON GROUND
1		.1	.01	16
2		TR	TR	16
3		TR	TR	16
4		TR	TR	16
5		6.6	.29	16
6		1.1	.05	23
7		1.9	.09	23
8		.8	.03	23
9		TR	TR	21
10		TR	TR	20
11		.4	.02	20
12				19
13				19
14		TR	TR	18
15		.4	.01	18
16	TR	.4	.02	18
17	TR	1.5	.08	19
18	TR		TR	18
19		.2	.02	15
20				15
21		.7	.02	15
22		.3	.01	16
23		TR	TR	16
24		TR	TR	16
25				16
26		.1	.01	15
27		4.3	.32	15
28	-	.6	.04	18
29				
30				
31				
Total	TR	19.4	1.02	
Normal	.01	7.6	.77	

THUNDERSTORMS		
NO.	NORMAL	DATES
0	0	-

DAILY WEATHER SUMMARY FOR FEBRUARY 1965 AT EDMONTON City.

1. Clear, cldy during day, light snow, gusty winds. Cold.
2. Clear, light winds, warmer by late aft.
3. Clear and very cold in mrng. Smk and fog. Cldy rest of day. Lgt wnds.
4. Overcast. Snow in mrng. Smk. Lgt wnds.
5. Cldy. Snow, ocnly hvy. Winds bcmg strong in eve, ocnly gsty.
6. Cldy xcpt clr in eve. Snow in mrng. Cold. Strong, gsty ovr ngt.
7. Mainly cldy. Mild. Snow. Mainly lgt winds.
8. Cldy. Snow. Warm. Lgt winds xcpt late evng.
9. Cldy. Snow erly mrng. Colder. Winds NWly and gusty erly mrng then lgt Sly.
10. Mstly cldy, snw flurries in the mrng. Cool. Winds lgt.
11. Cldy with snw late mrng. Warm. Wind lgt bcmg 17-26 & gusty in the evng.
12. Sunny bcmg cldy late evng. Mild. Winds lgt.
13. Cldy bcmg sunny late mrng. Warm. Winds 12-25 gusty aftn.
14. Mstly cldy with lgt snw early mrng. Cold. Wind 14-24 and gusty in the mrng bcmg lgt.
15. Cldy. Ocnl snw. Lgt winds.
16. Cldy. Warm. Smoke late mrng. Snow and rain shwrs late aftn. Winds mainly lgt.
17. Cldy. Warm. Snow erly mrng. Lgt winds.
18. Cldy. Fog most of day. Rain shwr late eve. Lgt winds. Warmer.
19. Overcast. Snow in aftn. Wind gusty late aftn & eve. Warm.
20. Colder. Gusty winds in aftn. Few clds.
21. Clr, bcmg cldy late mrng. Lgt snw aftn. Cold. Wind lgt bcmg 12-20 aftn.
22. Ovc with lgt snow bcmg sunny aftn. Cold. Wind lgt.
23. Clr bcmg cldy. Snw in aftn. Cool. Wind lgt.
24. Cldy. Lgt snw early mrng. Mild. Wind lgt.
25. Clr bcmg cldy in the mrng. Warm. Wind lgt.
26. Cldy snw late mrng. Warm. Wind lgt.
27. Ovc. Snw most of day. Cold. Wind 8-25 gusty aftn.
28. Ovc with snw, clrng aftn. Cold. Wind lgt.

TOTAL PRECIPITATION; NO. OF DAYS WITH:					SNOWFALL; NO. OF DAYS WITH:				
0.01" or more	0.04" or more	0.25" or more	0.50" or more	1.00" or more	0.1" or more	0.4" or more	2.5" or more	5.0" or more	10.0" or more
15	6	2	0	0	15	11	2	1	0

(1) T = Trace = negligible amount of rain or snow. Precipitation data are based on 24-hour period beginning 11 p.m. and ending 11 p.m. the following day.

~~(2) Total [illegible]~~

(3) Measurement taken at 0500 M.S.T., 1100 M.S.T., 1700 M.S.T. and 2300 M.S.T..

METEOROLOGICAL BRANCH - DEPARTMENT OF TRANSPORT - CANADA

MONTHLY METEOROLOGICAL SUMMARY

for

MARCH 1965

A RECORD OF OBSERVATIONS TAKEN AT EDMONTON, ALBERTA.

Date	TEMPERATURE (°F)			Degree Days below 65°F	RELATIVE HUMIDITY (%)				WIND (m.p.h.)			Duration of Bright Sunshine (hours)	Heat Units-Degree Days above 42°
	Maximum	Minimum	Mean		0500 M.S.T.	1100 M.S.T.	1700 M.S.T.	2300 M.S.T.	Average Speed from Anemometer	Prevailing Direction (by hours)	Maximum Speed for 1 hour with Direction		
1	29	-10	10	55	72	69	71	80	5.5	S	SW 9	1.7	
2	37	7	22	43	82	79	68	77	5.9	S	S 8	9.7	
3	44	16	30	35	83	71	59	77	6.8	S	S 11	9.4	
4	43	17	30	35	94	75	54	77	6.1	NW	N 10	8.6	
5	38	17	28	37	91	80	64	78	7.1	S	S 16	9.5	
6	45	22	34	31	87	68	51	71	10.6	WNW	S 17	9.5	
7	45	20	33	32	80	71	50	77	5.8	SW	NW 9	9.8	
8	40	27	34	31	72	68	59	67	13.7	NW	NW23	6.8	
9	39	19	29	36	81	80	74	77	7.8	N	N 14	0.0	
10	43	20	32	33	84	83	69	50	9.3	NW	NW24	7.0	
11	42	31	37	28	84	100	84	89	20.8	N	NW32	0.0	
12	33	22	28	37	82	77	71	83	9.8	N	N 15	9.0	
13	42	19	31	34	81	57	64	81	18.5	NW,N	N 34	6.8	
14	29	17	23	42	79	78	75	73	12.9	N	N 20	5.1	
15	17	6	12	53	84	80	69	68	8.5	NE	N 15	0.0	
16	10	- 6	2	63	71	65	72	58	9.7	NW	N,NW13	8.6	
17	13	- 3	5	60	73	69	46	63	9.0	NW	NW12	9.4	
18	12	- 4	4	61	75	68	54	58	9.1	SW	N 14	8.6	
19	13	- 6	4	61	71	59	28	69	5.8	W	N 9	9.9	
20	20	-11	5	60	75	53	58	71	5.4	S	W 10	7.0	
21	21	8	15	50	87	71	57	70	15.0	N	NW,N 23	6.0	
22	8	- 2	3	62	80	71	70	75	9.2	N	N 19	7.2	
23	8	-11	- 2	67	81	68	72	62	8.7	NW	NW14	9.2	
24	10	- 8	1	64	73	55	69	60	5.3	N	N 12	9.3	
25	9	- 8	1	64	62	60	69	63	7.0	NW	SW13	8.7	
26	10	- 2	4	61	74	66	71	73	9.6	SE	SE21	7.0	
27	11	- 2	5	60	84	76	62	75	9.1	E	E,SE12	5.6	
28	14	- 4	5	60	82	71	61	78	9.5	SE	SE16	9.0	
29	21	5	13	52	79	70	65	79	12.4	SE	SE16	4.5	
30	28	13	21	44	82	75	66	75	5.3	S	SE10	7.1	
31	40	10	25	40	77	61	47	74	7.7	SE	SE16	10.3	
Mean	26.3	7.1	16.7	Total 1491	79	71	63	72	9.3	Prevailing S	Maximum N 34	Total 220.3	
Normal	31.0	13.2	22.1	1290	-	-	-	-	8.9	S	30	166.2	

UDC: 551.506.1 (712.33)

PRECIPITATION (INCHES)				
Date	RAINFALL	ACTUAL DEPTH OF NEWLY FALLEN SNOW	TOTAL PRECIPITATION	TOTAL DEPTH OF NEW AND/OR OLD SNOW ON GROUND
1				18
2				17
3				15
4				14
5				13
6				13
7				12
8		TR	TR	11
9	TR	TR	TR	11
10		TR	TR	10
11	.07	3.4	.21	9
12	TR	TR	TR	11
13		.4	.02	11
14		TR	TR	11
15		.5	.03	11
16		TR	TR	11
17		TR	TR	11
18		TR	TR	11
19				11
20		TR	TR	11
21		1.2	.07	11
22		.7	.04	11
23		.2	.02	11
24				11
25		TR	TR	11
26		.1	.01	11
27		.5	.05	11
28		TR	TR	11
29		TR	TR	11
30		TR	TR	11
31				10
Total	.07	7.0	0.45	
Normal	0.05	7.8	0.83	

THUNDERSTORMS

NO.	NORMAL	DATES
0	0	-

DAILY WEATHER SUMMARY FOR MARCH 1965 AT EDMONTON, ALTA.

1. Few clds, lgt winds, mild.
2. Clr and mild, Hi 37, Lgt wnds.
3. Clr and mild, lgt winds.
4. Few clds, lgt wnds.
5. Clr, lgt wnds.
6. Clr, lgt wnds.
7. Clr, lgt wnds.
8. Cldy, strong wnds.
9. Ovcast, lgt wnds.
10. Clr, clding over by noon, strong wnds late evng.
11. Overcast with rain & snow, strong winds.
12. Snw in early morning then sunny but cool.
13. Sunny and strong winds in morning, clouding over in the afternoon with snow.
14. Cldy & cool. Winds 8-20 gusty early mrng. Lgt snw shwrs early mrng.
15. Cldy. Snow, lgt winds, Cool.
16. Cldy bcmg clr. Light snow. Lgt. Winds. Cold.
17. Few clds, lgt winds, snow early morn. Cool.
18. Cldy and Cool.
19. Clr & lgt winds.
20. Clr becoming overcast by noon, lgt wnds.
21. Cldy with snw in morning.
22. Cldy with snw.
23. Cldy with snw in early mrng clring by noon and sunny.
24. Sunny but cool.
25. Cldy, cool, lgt winds, snow in aftn.
26. Cldy, cool, lgt winds, lgt snow in aftn and eve.
27. Cldy with lgt snow, cool, winds lgt.
28. Few clds, lgt snow early morn. Lgt winds. Cool.
29. Overcast, lgt snw in morning.
30. Cldy snw in morning.
31. Sunny with strong winds in the evening.

TOTAL PRECIPITATION; NO. OF DAYS WITH:					SNOWFALL; NO. OF DAYS WITH:				
0.01" or more	0.04" or more	0.25" or more	0.50" or more	1.00" or more	0.1" or more	0.4" or more	2.5" or more	5.0" or more	10.0" or more
8	4	0	0	0	8	6	1	0	0

(1) T = Trace = negligible amount of rain or snow. Precipitation data are based on 24-hour period beginning 11 p.m. and ending 11 p.m. of the following day.

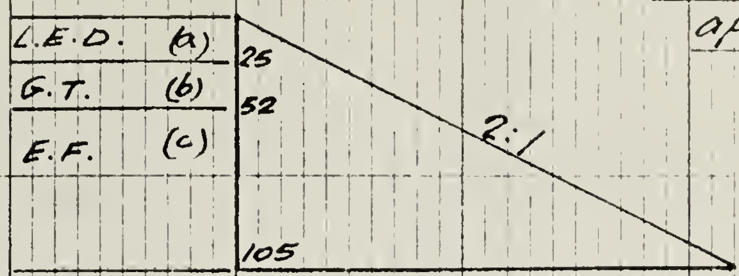
(3) Measurement taken at 0500 M.S.T., 1100 M.S.T., 1700 M.S.T. and 2300 M.S.T.

APPENDIX C

TYPICAL STABILITY ANALYSES OF LANDSLIDE

SIMPLIFIED SLIDING BLOCK ANALYSES
FOR APPROXIMATED PRE-LANDSLIDE SLOPE.

See Figure 14. for appropriate parameters.



(a) $\gamma_b = 120$ pcf. $H = 105$ ft.
(b) $\gamma_b = 130$ pcf.
(c) $\gamma_b = 114$ pcf.

Assesed Shearing Stress:-

$$120 \times \frac{25^2}{2} \times 0.49 = 18,375$$
$$+ 27\left(\frac{120\times 25 + 130 \times 52}{2}\right) \times .37 = 48,751$$
$$+ 53\left(\frac{130\times 52 + 114\times 105}{2}\right) \times .71 = 352,405$$
$$419,531 \text{ p.f.}$$
$$= \frac{419,531}{210 \times 2000} = \underline{\underline{0.99 \text{ t.s.f.}}}$$

Total normal Stress (σ).-

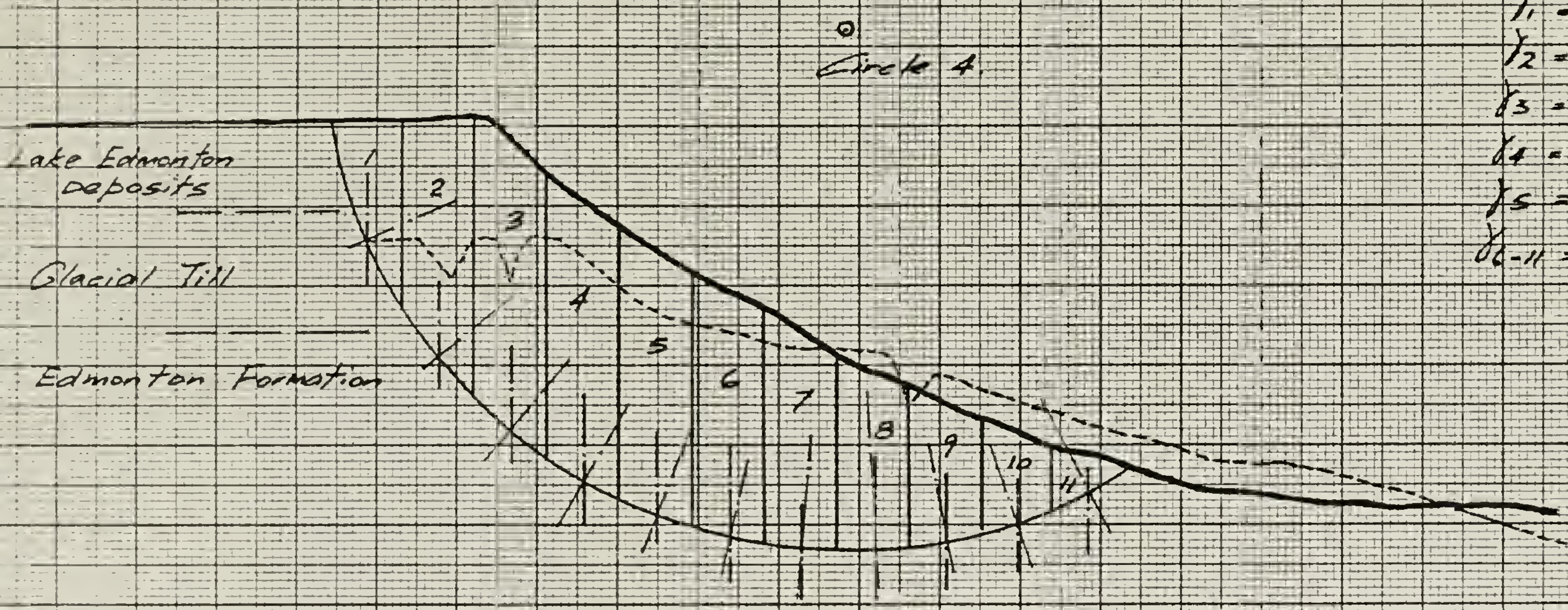
$$25\times 25 \times 120 = 75,000$$
$$+(52\times 52 - 625)130 = 270,270$$
$$+(105 \times 105 - 52\times 52)114 = 948,594.$$
$$1,293,864 \text{ p.f.}$$
$$\frac{1,293.9}{2\times 210} = \underline{3.1 \text{ tsf.}}$$

Pore Pressure.-
Linear Distribution = 58 ft
$$\frac{58\times 62.4}{2000} = \underline{\underline{1.8 \text{ tsf.}}}$$

Shearing Strength:-

Bentonitic Clay Shale. $c' = 410$ psf $\phi' = 18°$
$$S = .21\,(3.1 - 1.8)\,.33$$
$$= \underline{0.64 \text{ tsf.}}$$
$$F = \frac{0.64}{0.99} = \underline{\underline{0.65}}$$

Bentonite. $c' = 1741$ psf $\phi' = 4°$
$$S = .87 + (3.1 - 1.8)\,0.07$$
$$= \underline{0.96 \text{ tsf}}$$
$$F = \frac{0.96}{0.99} = \underline{\underline{0.97}}$$



Assumed:-

γ_1 = 122 pcf c' = 0 φ' = 22° — L.E.D.
γ_2 = 125 c' = 307 psf φ' = 26° — G.T.
γ_3 = 123
γ_4 = 122
γ_5 = 119
γ_{6-11} = 114

Average Parameters for Edmonton Formation
c' = 1536 φ' = 10°.

$$⑤ = \frac{\sec\alpha}{1 + \frac{\tan\phi' \tan\alpha}{F}}$$

Slice	Height H	Width b	α	W Sin α $\times 10^3$	① Weight $\times 10^3$	② c'b $\times 10^3$	③ W(1-r_u) tanφ'	④ ②+③	ΔH $\times 10^3$	⑤ Fs 1.0	⑤ Fs 1.13	④×⑤	④×⑤	u $\times 10^3$	r_u	tanφ'	(1-r_u)	tanφ'/1.13
1	30'	18'	66.5°	60.4	65.9	-	26.6	26.6	7.66	1.30	1.38	34.58	36.71	-	-	.404	1.0	.358
2	60'	18'	51.5°	105.7	135.0	5.53	56.7	62.2	7.50	1.00	1.04	62.23	64.72	1.06	.14	.488	.86	.432
3	73'	18'	40.25°	104.4	161.6	27.65	21.6	49.3	8.98	1.14	1.16	56.14	57.13	2.12	.24	.176	.76	.155
4	70'	18'	30.5°	78.1	153.7	27.65	18.1	45.8	8.54	1.05	1.06	48.04	48.50	2.81	.33	.176	.67	.155
5	67'	18'	21.5°	52.7	143.5	27.65	14.9	42.6	7.97	1.00	1.01	42.55	42.98	3.25	.41	.176	.59	.155
6	62'	18'	15.0°	28.6	127.2	27.65	11.6	39.3	7.07	0.99	0.99	38.86	38.86	3.37	.48	.176	.52	.155
7	55'	18'	5.25°	10.4	112.9	27.65	9.3	36.9	6.27	0.99	0.99	36.58	36.58	3.37	.53	.176	.47	.155
8	45'	18'	-3.0°	-4.8	92.3	27.65	6.2	33.9	5.13	1.01	1.01	34.19	34.19	3.18	.62	.176	.38	.155
9	35'	18'	-10.5°	-13.1	71.8	27.65	3.8	31.5	3.99	1.05	1.04	33.02	32.71	2.81	.70	.176	.30	.155
10	22'	18'	-18.75°	-14.5	45.1	27.65	1.0	28.7	2.51	1.12	1.11	32.09	31.80	1.31	.87	.176	.13	.155
11	9'	18'	-27.0°	-8.4	18.5	27.65	-0.9	26.8	1.03	1.23	1.22	32.90	32.64	1.31	1.27	.176	-.27	.155
			Σ	399.5							Σ	451.2	456.8					
											F	1.13	1.14					

ANALYSIS OF LANDSLIDE
ACCORDING TO BISHOP
(1954)